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Filtration for Chicago

THAT Chicago needs a clear, sparkling drinking water requires no stretch of the imagination, particularly during those months when Lake Michigan is so storm-tossed that turbidities in the hundreds are obtained even from its cribs farthest out in Lake Michigan. Few people in the office-building district drink anything but bottled water and trade in this commodity reaches almost unthinkable figures, for transportation of water by rail is far more costly than by city mains. As stated in this journal not so long ago the time has come to discuss filtration, and recently the three experts studying the future sanitary policy of Chicago reiterated this view. They believe that for the protection and improvement of the supply the city should look forward to the construction of filtration works. The present sanitary quality of the water is not questioned, but there is arising a demand for a clearer water than it is possible to obtain without filtration with cribs at any reasonable distance from shore.

Status of Electric Transmission

A SURPRISE awaits anyone of the opinion that only a few electric transmission systems have thus far been constructed for operation above 70,000 volts if he will turn to the "Electrical World" of April 25, in which a comprehensive tabular summary of fifty-four extra high-tension installations is set forth by Selby Haar. No less than thirty-five plants are now running at or above 100,000 volts working pressure, and a transmission from 100 to 200 miles long has become almost a commonplace as an achievement, although the details of the systems outlined are diversified enough to make close comparisons both interesting and difficult. The advance in this field in the last two or three years has been extraordinarily rapid and it is unlikely that the present maximum line pressure of 150,000 volts and the present maximum distance of transmission of 245 miles will be able long to retain their primacy, even though the difficulties for voltages above 150,000 are tremendous. Nearly 1,800,000 kw are installed in the stations listed and in many cases an ultimate development of two or three times the initial layout is provided for. Nearly all the plants are of the hydro-electric type, but the utilization of water power for high voltage transmission is usually so involved with the maintenance of reserve steam plant capacity that one cannot help wishing the latter might have been included in the data. Without personally visiting every system, however, and making laborious and detailed notes at great cost of time and effort, it is practically impossible to compile all the essential information bearing upon such modern plants, and if the view given is of the birdseye sort, the author has none the less performed a task of much difficulty and contributed substantially to the knowledge of the engineering industry.

Roads in Mexico

ALTHOUGH international problems such as the one awaiting solution in Mexico are of interest chiefly from the diplomatic or military viewpoint, there are certain phases which are within the province of the engineer. Last week, for example, the Engineering Record published a contribution by Surgeon-General Gorgas on sanitation in army camps, and the leading article in this issue deals with roads between Vera Cruz and Mexico City. If General Huerta dynamites the bridges on the railroad lines between the two cities, the roads and trails will be the only routes open to the United States troops in the event of orders to advance upon the Mexican capital. The air-line distance is about 200 miles and the country is characterized by extreme ruggedness. Obviously the building of roads with Indian labor under such conditions presents problems vastly different from those with which engineers in this country are familiar and the article by Guy Borden and Lucien G. Henderson, therefore, should be of considerable interest, particularly at present. Throughout rural Mexico the best roads are of inferior construction and in large measure are mere trails over which only pack trains may pass. These trails, however, are passable the year round. Over them a mule carrying a pack of 300 lb. will average 20 miles per day. The work involved in breaking a route through to the capital cannot be fully appreciated by those unfamiliar with construction in the tropics, with the type of labor available and the physical character of the country. The details given in the article in this issue will help to familiarize American engineers with the difficulties. The thought cannot be suppressed that an advance upon Mexico City, so glibly advocated by certain jingo interests, will probably be fraught with much hardship and severe loss of life.

Slandering Rival Products

COMMERCIAL honesty seems not, in the minds of some, to extend to the presentation of a case against a rival product. Men who would have conscientious scruples against stealing a dollar from a competitor will do him untold damage by showing the shortcomings of a product under certain conditions and concealing or denying that it has given general satisfaction under others. Recently there appeared in a journal devoted to the brick industry an article which would convince the unwary that concrete pavements were in all cases and under all conditions an utter failure. Thirteen views of wretched concrete pavements in Chicago were presented and early in the article was the sweeping statement that "the history of concrete roads and streets in Chicago is the same as that of any city or locality where concrete has been employed." Since the opening sentence of the article says that concrete streets in Chicago are a failure, it is fair to assume that the two statements taken together mean that concrete roads everywhere have been a failure—which is a plain falsehood. Let it be admitted that there have been bad concrete pavements, that most of those illustrated are wretched, that, if the facts warrant, concrete pavements in Chicago are a failure, and still the general indictment of concrete pavements is entirely unwarranted. Furthermore had the author of the article and the editor of the paper been well informed or applied stricter standards of commercial honesty, they would have admitted that the pavements shown had not been properly maintained. The patching of pavements with pitch is not new and its application to concrete roads should give quite satisfactory results. Pushing the idea of commercial honesty still further these horrible examples of concrete road failures should not have been shown when it was known that equally bad examples of brick pavement could be found, and this in the face of the fact that experience with brick has been longer than with concrete and that practice should be correspondingly better. While unfair presentations of this kind influence certain weak minds, fortunately the larger number of engineers do not base their decisions on what is obviously a biased and narrow investigation. Eventually such tactics hurt those who employ them.

Social Features in Local Societies

RECENT visits by members of the staff of this journal to many of the local engineering societies throughout the country have shown that there is a decided tendency, not evident four or five years ago, to emphasize the social side. Whereas formerly the societies were purely technical bodies, jealous that nothing outside of engineering matters should enter their deliberations, now there are efforts to bring the members together on a friendly basis monthly or even weekly. A smoker or dinner once a year was heretofore regarded as sufficient social diversion. The Western Society of Engineers is now endeavoring to have a good fellowship gathering after each meeting; the St. Louis Engineers' Club held a picnic last year, at which a clever morality play written by Mont Schuyler was presented, while not a few societies have arrangements whereby those members who wish may lunch together once a week. San Francisco, with truly Western spirit, carries out the social feature with thoroughness. Reporting informally on the meeting of the San Francisco members of the American Society of Civil Engineers on April 17, a correspondent says: "The association here plans its meetings on the theory that the informal 'mixing' is the most valuable part of the program, an idea which is as good as it is rare. Members go directly from their offices to a 6 o'clock supper. This is immediately followed by the few informal talks that the master of ceremonies sees fit to call for, and then a recess of half an hour or so is declared, during which everyone talks shop, and strangers are taken around by a 'mixing' committee. After this the paper of the evening is announced. The paper last night was illustrated by moving pictures as well as ordinary slides." The idea of the "mixing" committee might well be adopted by all local, and even by national societies, for nothing is so disheartening to a visitor as to be left in the company of just one member for the whole evening. He probably has called that he may get acquainted. The officers of the societies visited were practically unanimous in declaring that the expansion of the social features has been beneficial in cementing friendships and in increasing the membership. Such being the case there is reason to expect that societies which still are stiffly technical will soon try to bring out the human side of their members.

Air Vents on Pipe Lines

AIR vents on pipe lines seem to be among the neglected factors in hydraulic design. If the pipe is computed to resist collapse under a pressure difference equal to the full atmospheric pressure acting from without no air vents are necessary, but computations are seldom made on this basis. Moreover, it would be uneconomical to design for this condition.

In dimensioning air vents engineers generally depend upon rough guesses instead of careful computation. Accidents to pipe lines, therefore, are not infrequent. What is more, many lines now in "successful" operation stand up only because they have never been called upon to resist stresses due to emergencies of this nature.

In this issue, on page 594, there is an article by M. L. Enger and F. B. Seely, pointing out the necessity of adequate provision for air vents. It deserves the attention of every hydraulic engineer. The authors conclusively demonstrate the chances of disaster that are being taken in a number of installations.

On penstocks and inverted siphons air vents are generally provided at the upper end only. The mere provision of vents at the upper end, however, must not be considered as disposing of the collapsing problem. It must be borne in mind that the pipe is subject to stresses from without even for velocity changes not necessarily creating what is commonly termed a "vacuum." The collapse of the Antelope Valley siphon of the Los Angeles Aqueduct was due to just such a velocity change. Apparently, then, the pipe line must be designed to resist stresses due to these changes.

Air vents at the top of penstocks are seldom or never adequate. Two recent examples can be cited to supplement those discussed by Messrs. Enger and Seely. There was described in this journal recently a development in which 18-in. air vents were provided on 8-ft. penstocks. Should the penstocks in this installation break near the power house and the area of the rupture equal that of the penstock, air must enter the vent at a velocity of about 2000 ft. per second to prevent the total collapse of the line. Moreover, a 9-ft. penstock provided with a 20-in. air vent collapsed within a few months. Hasty calculations show that the velocity change was sufficient to cause the damage, and, that for a break near the power house, the air must either have entered through the vent at a velocity of 1000 ft. per second or the pipe have collapsed.

The examples cited here and in the article are but a few from many that could be selected. They show the importance of giving greater attention to this neglected problem in pipe-line design.

New Grading Rules for Yellow Pine Timber

RECENT discussion on the quality of yellow pine timber for mill construction has conclusively demonstrated that existing specifications are unsatisfactory. Grading, heretofore, at least in the standard specifications of engineering societies, has been based on the percentage of heartwood showing; on defects and, to some extent, on their location, and, for the highest class of timber, on segregation of longleaf from shortleaf pine. The attempt has been made to secure a high-strength wood by means totally inadequate. Timber might meet the specifications of the American Society for Testing Materials, for example, yet be wide-ringed, and have a very low percentage of dense summerwood. Many purchasers, realizing the inadequacy of the standard specifications, have bought under "rings to the inch" clauses, but even they, generally, have neglected the relative proportions of springwood and summerwood.

Now comes the Yellow Pine Manufacturers' Association with a specification notable for two reasons—in that strength quality is made the primary basis for the grading, and that permissible defects have been localized in a logical manner. Having received the approval of the board of directors the new classification is almost certain to be adopted by the association next July. It will be found on page 579.

The specification does not distinguish between longleaf and shortleaf pine, relying on the detailed clauses to bar out undesirable material. Density—in other words, strength—is secured by specifying that there shall be eight rings to the inch, containing in the greater number of rings 25 per cent or more of summerwood. Certain modifications are made, allowing fewer rings, provided the percentage of summerwood is higher. Permissible defects are localized with reference to the stresses to be carried; the allowance for posts is, for obvious reasons, more liberal than for stringers.

Just what the effect of these specifications will be is difficult to foretell, but it would be surprising if they did not result in the rewriting of the standard specifications of various associations. There has been dissatisfaction with the existing rules and a ring clause has long since been foreshadowed. In the lumber-producing world the new classification has made a decided impression, and there is reason to believe that the Georgia-Florida association of yellow-pine manufacturers will follow the lead of the older association. It should be noted that a producers' association had the courage to adopt the new grading, without waiting to have it forced on them by the engineers and users. This offers hope that the final step, branding, advocated in this journal, will come without great difficulty. It is true, of course, that the new classification makes branding less important than before, but it would protect those who will not take the trouble to adopt the more scientific grading and go to the trouble of inspecting shipments carefully.

The Way of Progress

A DOG barking at a passing automobile is generally supposed to be as telling a symbol of futile objection to the march of progress as could well be imagined. In almost the same category, however, belongs the strike of the stevedores in New Orleans against the introduction of the electric truck to transport freight between vessels and warehouses. The wonder is that this improvement has been so long delayed instead of only now appearing—and then as a source of a new labor difficulty. One cannot have much sympathy for opposition in this particular instance. The motor vehicle in all its forms has come to stay, and the rest of the world has been rather rapidly adjusting itself to the new condition. The carriage maker now builds automobile bodies, a little bicycle repair shop has become a two-story garage, and even the farmer, objurgating his frightened horse, has ended by selling the animal and buying an automobile himself. It may safely be predicted that the stevedores' strike will wind up without serious trouble. The number of men is too small to find much difficulty in the search for other work of a comparatively rough character.

Just one suggestion of this latest revolt against machine methods should not, however, be passed over without comment. It has almost always been assumed as a fixed reference point in political economy that labor-saving devices, through cheapening of product, open new and profitable avenues for labor to those who are displaced by the initial change. It is not altogether clear that in the modern organization of business and society this optimistic view is quite sound. It is no longer true that cheapening production lowers prices to the consumer to an adequate degree for the purpose in hand. Further, most great displacements of labor by machinery have come on very gradually during a period when the world still had ample and near frontiers to absorb the unemployed. It is not safe to assume that the same economic conditions will hold if prices are allowed to be controlled by trust agreements rather than by the cost of production, if competition is systematically eliminated, and when new lands are much less easily available than they were at one time. Governmental activity promises, however, to put a stop to the first two abuses.

The stevedores in New Orleans seem to be in their attitude the relic of a less adaptable age than our own, and while one may be sorry at their plight and wish them good fortune in obtaining other employment, it is rather ludicrous to see a strike on such grounds in the present day.

Training for Public Service

THE first function of popular education is to awaken people to their environment, next to enable them to adapt themselves to it. If the educational means provided do not do this they are imperfect to that extent. If the educators and the educated realize the shortcomings of the educational system there is hope for improvement. As our system is under most vigorous fire at present it is evident that there are glaring defects in the system and that we realize them more or less.

The Engineering Record is moved to take up this subject editorially because the need for a better quality of leadership is so acute. This is particularly true in public service. The stagnation in business has been produced not by crop failure or other national disaster but by lack of confidence, engendered by politicians incapable of getting beneath the surface and solving our problems on basic grounds. There are hopeful signs, however, signs that we appreciate that the difficulty is largely one of combining theory with practice, for educators and leaders in business and public life are getting together as never before.

To illustrate the last statement the conference called in New York City last week by Mayor Mitchel, under the auspices of the American Political Science Association, may be cited. This meeting was for the purpose of furthering the cause of practical education for public service, of helping the teachers to appreciate the problems of practical political life and of making provision for bringing students into contact with that life. Mayor Mitchel is also sending out a commission to study co-operative training of all grades in Cincinnati, Wisconsin and elsewhere. What a shudder must have passed through the frames of New York's leading politicians of the past at the sight of a mayor actively aiding in the development of competent public servants. Yet the case is not isolated, for on an allied mission bent the Mayor of Philadelphia, Rudolph Blankenburg, last year journeyed to the University of Wisconsin with members of his municipal cabinet.

In the severe arraignment of our educational system the colleges are coming in for their share. Even so successful an educator as President Humphreys of Stevens Institute of Technology is reported to have said in a speech delivered in Philadelphia this week that college men, particularly political economists, are largely responsible for present conditions. This he attributes largely to the laying of undue emphasis on theoretical considerations. Our present situation in Mexico is attributed by many solely to the exaltation of the theoretical and the neglect of the practical aspects of political situations. And yet we cannot get away from the fact that the college is the keystone of the educational arch. But the colleges must meet the needs of the times, producing virile, keen, far-seeing, diplomatic leaders. Furthermore, the colleges cannot be blamed if they turn out some graduates or non-graduates who do not make conspicuous successes. There is no magical educational process which can make something mental out of nothing. Nor can the college be held responsible for the vagaries of its graduates. The papers have emphasized the fact that a labor agitator who has made himself conspicuous of late is a Harvard graduate. Nevertheless, Harvard certainly stands for law and order and has produced many leaders of the type which the country needs.

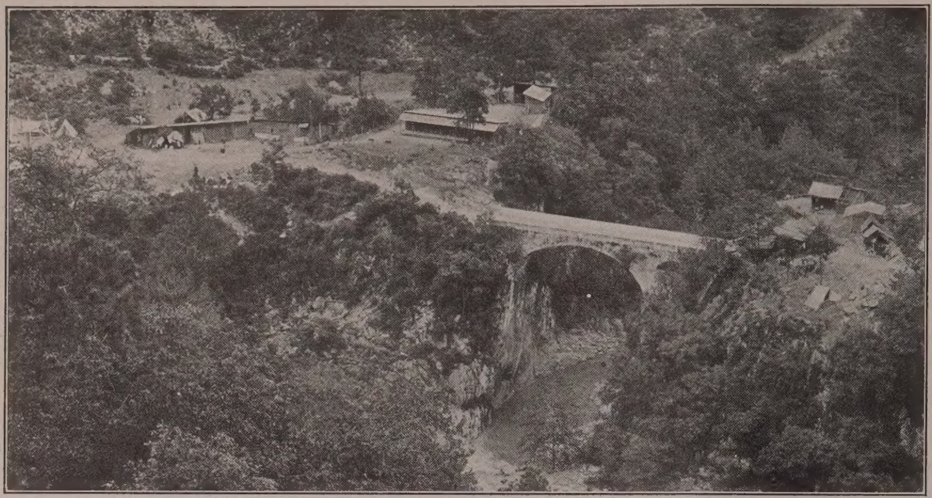
The advance of trained leadership and its displacement of the less efficient and sometimes self-centered variety is essential for the social welfare. Trained men are needed for public service commissions, for public service corporations, for city managerships, for many less conspicuous positions. The public utilities now suffering from the reaction of past offenses and from inadequate understanding of present circumstances on the part of the public will welcome the stable condition of our social fabric which can only be brought about under intelligent, unselfish leadership.

Eliminating a Drawback to Single-Phase Electrification

THE work recently done by the New Haven Railroad in the improvement of its electrical supply system is noteworthy as in the main removing one of the somewhat serious objections to the single-phase motor system. The lines of the New Haven are paralleled by telegraph and telephone lines, to a considerable extent on the same right-of-way and following the road through the entire electrified territory. As the railway system was originally installed it consisted in effect of an 11,000-volt trolley wire and feeders, with return through the rails. This system made in effect a gigantic magnetizing circuit consisting of a vertical loop having the trolley wire on one side and the rails on the other. The magnetic field from this reached out and cut every circuit in the vicinity, with the consequent production of inductive disturbances of greater or less severity. Under such circumstances a troublesome disturbance may reach a distance of several miles on each side of the line, provided the circuit with which the railway field is interlinked has sufficient extent. A very long loop furnished by a grounded telephone wire, for instance, may be disturbed at a considerable distance just as effectively, in fact, as if it were on the same right-of-way with the railway but inclosed a smaller area.

The usual method taken to dodge these electromagnetic disturbances has been by using a complete metallic circuit for the telephone or telegraph wires and transposing the conductors at frequent intervals so as to break the long loop into a succession of short and narrow ones in which the inductive forces would be alternately in opposite directions so as to neutralize them. This, if very carefully carried out, is reasonably effective, but on grounded circuits, of which there are still a good many left, the inductive trouble is serious, and even on the transposed circuits it is hard to get a degree of equality which will insure complete freedom from interference. Various ingenious compensating devices have been tried on the telephone and telegraph circuits, necessitating considerable trouble and expense. The step now taken, however, is to compensate the disturbing field so as to minimize the trouble from induction. This is done by splitting up the overhead system of the railway into sections by throwing auto-transformers, of two to one ratio of transformation, across the trolley and feeder circuit, the middle points of the transformers being connected with the track. The fields from the feeders and trolley wires thus tend to neutralize each other and the long inducing loop carrying the aggregate current of the system is broken up into sections each of which is approximately neutralized as regards the inductive field produced on neighboring wires. It is a very ingenious and apparently successful attempt to produce the result of a frequently transposed transmission line without the inconveniences which would result in trying to work a transposed overhead trolley system. An ordinary three-phase transmission line can be spiraled to lessen inductance very easily, but obviously the same scheme cannot be practically carried out on a system consisting of track, trolley wire and feeders.

The result of the new arrangement, which has now been in operation about three months, has been very gratifying. The disturbances due to the railway system have been greatly reduced so that the somewhat troublesome compensating arrangements tried on the telephone and telegraph wires have been greatly simplified. Incidentally it should be mentioned that while the new arrangement with its auto-transformers has been expensive, it actually is cutting down the cost on new construction, owing to the fact that the transmission voltage applied to the auto-transformers is 22,000 instead of 11,000. Thus two birds have been killed with one stone —first the inductive disturbances, which were of a most exasperating character, and, second, the unnecessary cost of the 11,000-volt circuit, which seems desirable for the trolley contact while too low for conditions of maximum efficiency in the transmission itself.



Type of Highway Bridge Commonly Encountered in Mexico

Roads between Vera Cruz and Mexico City

Types of Construction and Transportation Difficulties along Pack Trails in Rugged Country between Coast and Mexican Capital

By GUY BORDEN
Formerly Resident Engineer on Laxaxalpam Tunnels at Necaxa, Puebla, Mexico

and

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Photographs by courtesy of Jacobs & Davies, Inc., New York

EVEN the traveler who has several times made the railway journey from Vera Cruz to Mexico City and has appreciated thoroughly the difficulties overcome by the railroad engineers in the construction of these lines has little conception of what it means to make the trip by wagon road or trail. But with the prospect of this region becoming the scene of military operations, and the ease with which the destruction of a few bridges or tunnels could make the railways useless, the highway route, now half forgotten, may become of prime importance. It is easily conceivable that an invading army to-day might face transportation problems similar to those encountered by the army of General Scott in 1847 and the French in 1862. The mountainous nature of the country to be traversed would absolutely preclude speedy passage of an army in any other way than over definitely constructed roads.

At the start it must be clearly realized that continuous paved highways, as American engineers understand the term, do not exist in the Republic of Mexico, except for a few short stretches in the immediate vicinity of some of the larger towns. Engineers who have been over the roads from Mexico City to Toluca on the west and Cuernavaca to the south—conspicuously the best roads running into the capital—will remember them as distinctly second-rate highways. This unimproved condition of the roads exists throughout all rural Mexico, although even the smaller towns are usually well paved with cobblestones, while the asphalted city streets compare favorably with the boulevards of the United States and Europe.

ORE ROADS

The fact that road building has not kept pace with the industrial progress of the country in other lines is easily traceable to economic causes, for in Mexico, as in the United States, railroads were built before agricultural developments on a large scale took place. What roads were constructed before the railroads came were for the purpose of connecting the larger mining centers with Mexico City or with the coast, for the transportation of ore, machinery and supplies.

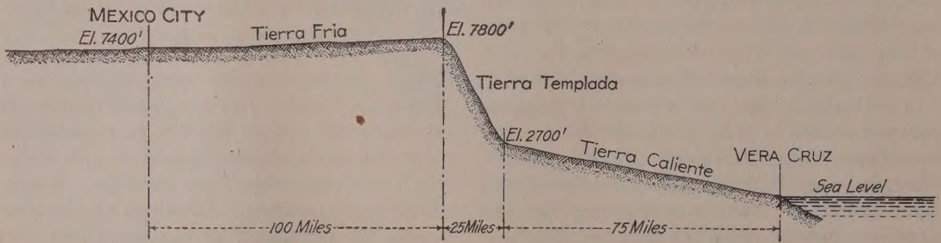
DURABILITY OF OLD ROADS

In their best days some of these ore highways must have been fully equal to any roads then existing in the United States. Across marshy lands one can still see long stretches of earthen causeways, with remnants of their cobblestone paving still in place. In the mountains heavy cuts evince a thoroughness in road construction that the Spanish builders must have inherited from Roman times; even now old stone bridges on which are carved the Spanish coat of arms are in almost as good condition as when their builders finished them centuries ago. Of such a character are the "caminos reales," or royal high roads, that connect Mexico City with the port of Vera Cruz—one through Puebla and Orizaba and the other by way of Jalapa. Before the completion of the first railroad in 1873, the Jalapa road was the "diligencia," or stagecoach route, between Mexico City and the coast. The journey, barring a hold-up by bandits, required three days under the most favorable conditions.

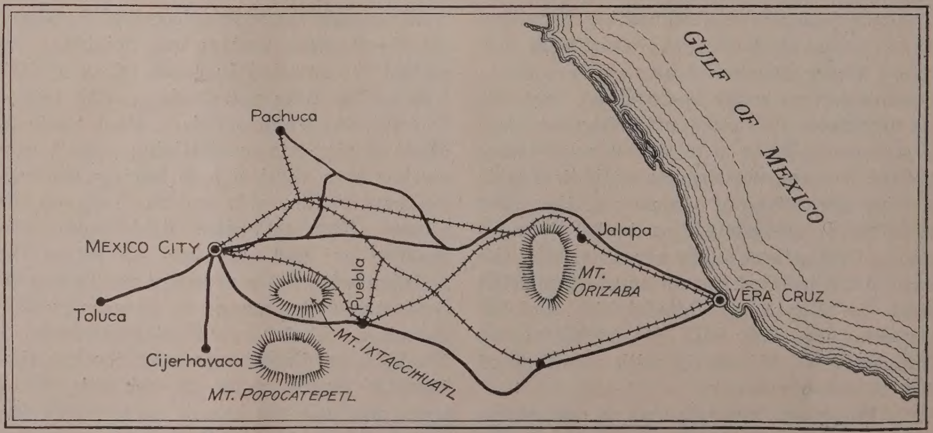
TOPOGRAPHY OF THE REGION

The air-line distance from Vera Cruz to Mexico City is about 200 miles, and any direct road between the two places has to overcome practically identical topographical difficulties. Mexico City is built on a plateau about a mile and a half above the sea. This great tableland extends nearly across the continent, its eastern edge being about 100 miles from the Gulf. The sketch gives a rough idea of the general conditions. From Vera Cruz a coastal plain, the "tierra caliente," some 75 miles in width, gradually ascends from sea level to an elevation of about half a mile. This is succeeded by an escarpment which rises a mile higher in a horizontal distance of 25 miles, the "tierra templada," reaching the elevation of the central Mexican plateau, or "tierra fria." The remaining 100 miles to the city of Mexico are over practically level tableland.

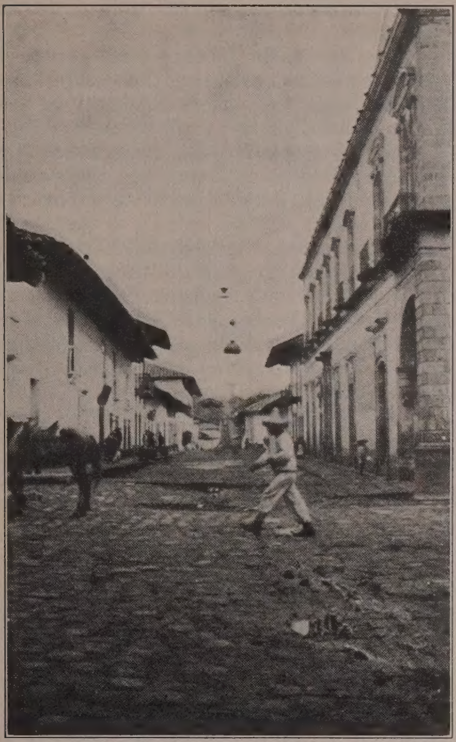
The coastal plain, although broken by foothills and traversed by numerous rivers, presents no abnormal road-building problems, nor do the 100 miles from the edge of the plateau to Mexico City. The difficulties of the route lie in the 25 miles of escarpment. This is a zone of almost inconceivable roughness. The seepage from the tableland above, issuing as springs, forms countless rivers flowing toward the sea; each of them has eaten its steep-sided gorge back into the escarpment. Between the gorges are huge "hogbacks," with their sides in turn cut up by precipitous ravines,



Profile of Ground Lying between Vera Cruz and Mexico City



Plan of Means of Communication between Vera Cruz and Mexico City



A Typical Mexican City Street

the channels of streams tributary to the larger rivers. And in the rough sides of these familiar "barrancas" of Mexican scenery every imaginable twist and fold of violent volcanic upheaval is discernible. Wherever not too precipitous for trees to take root the hillsides are heavily wooded, with pines and oaks in the higher altitudes and tropical vegetation below. The yearly rainfall is from 100 to 150 in., most of which occurs in the season from June to October. This heavy concentration of rainfall greatly augments the stream flow in this season of the year, so that what have been insignificant trickling streams in winter become torrents in July and August.

PRESENT CONDITION OF ROADS

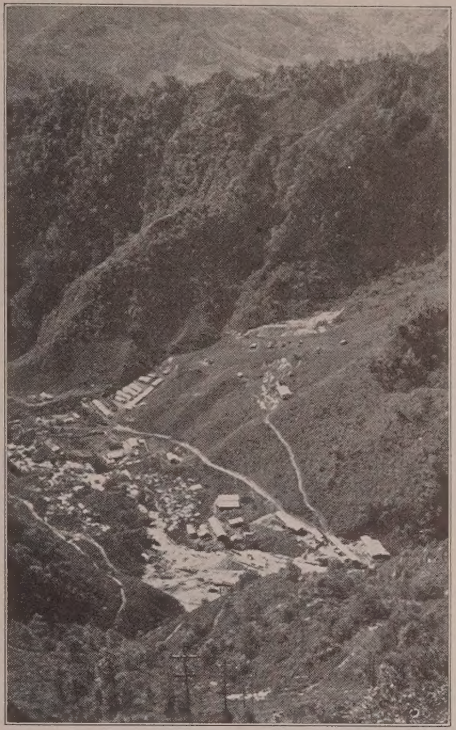
The roads which have been built from the coast to the plateau zigzag up the sides of these "hogbacks." In most cases they are sidehill cuts, with a wall of rock above and a sheer descent below. The usual width is about 15 ft., and the grades are as steep as a wheeled vehicle can be drawn over, 12 per cent not being uncommon. What little paving was placed originally was cobblestone work, but this now merely serves to keep a vehicle from bogging in the muddiest stretches, and in the mind of the modern engineer there would be a serious question as to whether these cobblestones, in their present state, are a help or a hindrance. But for the most part there is no surfacing whatever. The roads serve the purpose for which they are used at present, the passage of pack trains, but for vehicles hauling over them it would be difficult in the extreme. Recently one of the writers had occasion to go by automobile over about 35 miles of one of these old roads which was in constant use as a freight trail, and found it almost impossible to complete the journey.

The bridges, whose solidity and structural excellence excite the admiration of the engineer to-day, are all arches of stone, laid in lime mortar. Forty-foot spans are not uncommon, and in at least one case known to the authors a railway is carried by one of these old arches without additional reinforcing of any kind.

Three years ago one of the writers was concerned with the construction of a 16-ft. wagon road through this very region, quite similar in character to the roads above described, ascending from an elevation of 5000 ft. to one of 9000 ft. in about 6 miles. Through its entire length the road was practically a sidehill cut. Provision for drainage was made, but was of the simplest type, the road surface being sloped to a ditch along the inner edge, which discharged through cross-drains put in at frequent intervals. Although no surfacing was done, no bridges were built, and the resulting road is serviceable only in dry weather, the cost of this piece of work, with native labor at 50 cents per day and dynamite liberally used, was more than $1600 a mile. With all the labor that could be obtained locally the work took five months to complete. From these figures one can get a fair idea of what it means to build a road of even the simplest kind from the coastal plain to the plateau. Putting the old roads in fair condition would be proportionately difficult and expensive.

PACK TRAILS

Besides the wagon roads, the other means of transportation, and the one most widely used to-day, is the pack-trail. This part of Mexico is thickly populated by Indians of the Totonaca and Mixteca tribes, who cultivate small farms on the steep, rocky hillsides. Little towns and villages abound, each with its group of thatched huts clustered around a red-roofed, white adobe church. For communication between these settlements the inhabitants have constructed an elaborate system of mountain trails, which afford passage for Indian pack bearers and burro trains, but are out of the question for wagons or even carts. The trails are made no wider than is necessary—so narrow in most places that animals going in opposite directions cannot pass each other. The grades are incredibly steep and the path is frequently thickly strewn with boulders.



Meandering Roads along Tierra Templada

Where the trails cross dangerous streams there have been built stone arch bridges often equal in workmanship to the bridges on the wagon roads. A peculiar feature sometimes met with in these bridges is a steep grade in the roadway from each end toward an apex in the center, which avoids filling over the spandrels.

CONSTRUCTION BY INDIANS

Bridges and trails are constructed and maintained by what is known as the "faena" system, which requires every able-bodied man to put in several days a year in roadwork. It is not unusual to see two or three thousand bare-footed Indians, clad in the universal blue woolen shirt and white "pantalones," strung out over a quarter of a mile, building trail with their primitive agricultural implements and doing just as little work, of course, as their "padrones" will allow.

The system, crude as it is, seems to work fairly well. The trails are passable the year round, and over them a mule carrying a pack of 300 lb. will average 20 miles a day. A burro carrying about half that load makes about 15 miles. The Indian pack bearer is extremely efficient, and with a 150-lb. load strapped

Mexican Field Party in Charge of American Engineer

The man on the right, besides being an axeman, is a judge in his home town.

to his head with a "mecapae," or forehead band, can walk up a mountain trail so steep that an unhampered gringo hesitates to attempt it. In all seasons any part of the region is easily accessible to a man on horseback.

The two old highways, now practically useless for vehicles, and the mountain trails together constitute the overland route from the capital city to the coast—difficult, dangerous and passable only by lightly loaded pack trains. The country to be traversed is one of the roughest on the North American continent. If in the present crisis it becomes necessary to reach the city with the railroads out of commission, it will be interesting to see how our army handles the problem.

Rules Governing Architectural Competitions

Typical Form of Agreement between Owner and Competitors Required by American Institute of Architects

IN the Engineering Record of May 2, 1914, page 499, there was reference, in the general discussion on competitions, to the rules of the American Institute of Architects. The rules were amended in December, 1913, and have recently been issued in pamphlet form. They go into all details, but the gist of the institute's attitude is summed up under the caption of "Approval of the Program" and the typical form of agreement between owner and competitors. This form, printed below, is a contract to give a commission to one of the competitors. Fake competitions are, therefore, barred. The summation and agreement form are as follows:

Participation by Members and Approval of Program

Members of the American Institute of Architects do not take part as competitors or jurors in any competition the program of which has not received the formal approval of the institute, nor does a member continue to act as professional adviser after it has been determined that the program cannot be so drawn as to receive such approval.

The approval of the institute is not given to a program unless it meet the following essential conditions:

(a) That there be a professional adviser.

(b) That the competition be of one of the forms described in Article 3. [These forms are "limited" competitions, in which only certain architects are allowed to compete, and "open" competitions of a restricted character, in which, as a first stage, any architect can submit simple outline drawings, accompanied by evidences of education and experience. From those submitting designs a certain number are selected for the final competition.]

(c) That the program contain an Agreement and Conditions of Contract between architect and owner in conformity with those printed in the appendix [reprinted below] of this circular, that it include no provision at variance therewith, that it contain terms of payments in accord with good practice, and that it specially set forth the nature of expert engineering services for which the architect will be reimbursed.

(d) That the program make provision for a jury of at least three persons.

(e) That the program conform in all particulars to the spirit of this circular.

Form of Agreement

The form of agreement between owner and competitors is as follows:

In consideration of the submission of drawings in this competition (here insert the name of the owner or of the body duly authorized to enter into contracts on behalf of the owner), hereinafter called the owner, agrees with the competitors jointly and severally that the owner will, within...... days of the date set for the submission of drawings, make an award of the commission to design and supervise the work forming the subject of this competition to one of those competitors who submit drawings in consonance with the mandatory requirements of this program, and will thereupon pay him, on account of his services as architect, one-tenth of his total estimated fee as stated below. And further, in consideration of the submission of drawings as aforesaid, and the mutual promises enumerated in the subjoined "Conditions of Contract between Architect and Owner," the owner agrees and each competitor agrees, if the award be made in his favor, immediately to enter into a contract containing all the "Conditions" here following, and until such contract is executed to be bound by the said "Conditions."

Conditions of Contract Between Architect and Owner

Article I—Duties of the Architect

1. Design.—The architect is to design the entire building and its immediate surroundings and is to design or direct the design of its constructive, engineering and decorative work and its fixed equipment and, if further retained, its movable furniture and the treatment of the remainder of its grounds.

2. Drawings and Specifications. — The architect is to make such revision of his competitive scheme as may be necessary to complete the preliminary studies; and he is to provide drawings and specifications necessary for the conduct of the work. All such instruments of service are and remain the property of the architect.

3. Administration.—The architect is to prepare or advise as to all forms connected with the making of proposals and contracts, to issue all certificates of payment, to keep proper accounts and generally to discharge the necessary administrative duties connected with the work.

4. Supervision. — The architect is to supervise the execution of all the work committed to his control.

Article II—Duties of the Owner

1. Payments.—The owner is to pay the architect for his services a sum equal to 6 per cent[*] upon the cost of the work. (The times and amounts of payments should be here stated.)[†]

2. Reimbursements.—The owner is to reimburse the architect, from time to time, the amount of expenses necessarily incurred by him or his deputies while traveling in the discharge of duties connected with the work.

3. Service of Engineers.—The owner is to reimburse the architect the cost of the services of such engineers for heating, mechanical and electrical work as are specifically provided for in each program. The selection of such engineers and their compensation shall be subject to the approval of the owner.

4. Information, Clerk-of-the-Works, Etc. —The owner is to give all information as to his requirements; to pay for all necessary surveys, borings and tests, and for the continuous services of a clerk-of-the-works, whose competence is approved by the architect.

Preliminary Report on Sanitary Policy of Chicago

Experts Recommend Continuation of Sewage Dilution and Immediate Removal of Suspended Matter, with Future Intensive Treatment

FILTRATION of the water is the only way by which protection and improvement of the supply for the city of Chicago can be obtained. Furthermore, diversion of the sewage into the drainage canal and extension or alteration of the location of the intake would not result in drinking water of suitable quality. These conclusions are presented in the preliminary report of Dr. George A. Soper, John D. Watson and Arthur J. Martin, board of experts to the harbor and river-improvement committee of the Chicago Real Estate Board, following a study of the sewage-disposal problem on the ground for two weeks. Every opportunity was embraced for inspection, examination of existing data and personal interviews with Federal, State, city and Sanitary District officials and with private citizens. Abstracts of the ten-page preliminary findings follow.

Conditions

The conditions of sewage disposal which exist in the Chicago River and its outlet are conceded to be unsatisfactory, and it is the opinion of the board of experts that the disposal of the sewage cannot be made to answer the reasonable demands of sanitation without radical improvement.

It may be said that nuisances exist in various places in the contributing arms of the Chicago River, but not to a considerable extent, however, so far as the personal observations of the board of experts have extended, in the main channel from the lake to the drainage canal. Practically the entire watershed of the Chicago River is populated or soon will be built up. The drainage channel flows through open country for many miles, and its condition, although observed to be foul by those who have had occasion to visit it, does not constitute a nuisance for the reason that none are compelled to reside or work within the sphere of its offensiveness.

Vital statistics of Chicago have been compared with those of large cities whose drinking-water supplies have been filtered, and it appears to have been shown that Chicago has one of the cleanest and purest waters of any large city in America. There seems generally to have been sufficient protection afforded up to the present time. Apparently the object in placing the intakes as far out as practicable into the lake has been not only to avoid contamination along the water front but to get clear water. At times of storm the water of Lake Michigan becomes turbid near shore, clay being conspicuously present. After fresh easterly winds even the best water obtainable from the 4-mile crib becomes noticeably turbid.

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[*] Some competitions may be for work in which all or certain parts command a higher percentage, and in such cases this should be named in the contract.

[†] Good practice has established the payments on account as follows: Upon completion of the preliminary studies one-fifth of the total estimated fee less the previous payment; upon completion of contract drawings and specifications two-fifths additional of such fee; for other drawings, for supervision and for administration, the remainder of the fee, from time to time, in proportion to the progress of the work.

On the days when the inspections of the Desplaines and Illinois Rivers were made the water was offensively polluted with sewage as far as Ottawa—85 miles from Lake Michigan. Where bridges crossed the stream it was possible to see unmistakable evidences of the sewage. Odors were noticeable, grease lay upon the surface of the river, dead fish were numerous and large quantities of sewage fungus were observable in the water. Through the whole course of the stream from Chicago to and below Ottawa gulls were seen feeding on the solid refuse which the sewage brought down.

Opinions and Suggestions

Dilution alone cannot be relied on as a permanently satisfactory method of disposing of the sewage of the District. It offers an insufficient means of disposing of the sewage at the present time, and would prove still more inadequate in the future, in spite of the full quantity of water which the drainage channel can carry. It will ultimately be necessary to treat practically all sewage before it is discharged. The conditions which now exist are not so insanitary as to call for improvements which will involve immediate and large expenditures of money, but the preparation of comprehensive plans for sewage disposal, water supply and river development should be undertaken immediately.

It is not possible to procure a satisfactory water supply by diverting the sewage to the drainage channel and extending or altering the location of the intakes. For the protection and improvement of the supply the city should look forward to the construction of filtration works.

Some of the eighteen principles and lines on which to proceed are:

The principle of diverting the sewage from the lake should be adhered to, in spite of any degree of sewage purification or water purification which may be employed.

Full advantage should be taken of as large a volume of diluting water as may be obtainable from the lake, in order that the cost of finally disposing of the sewage may be reduced to a minimum.

It is desirable to concentrate as much of the sewage as practicable at a point beyond the built-up sections of the city and in the vicinity of the drainage channel, where the sewage can be treated sufficiently to permit the effluent to be discharged without serious danger of offense.

To avoid excessive cost and in order to relieve the drainage channel as far as practicable, it will be desirable to treat the sewage of some parts of the city in the areas in which the sewage is produced.

Practically no sewage should be discharged into the Chicago River or the drainage channel without some form of treatment, except in times of storm, when suitably located overflows will be permissible. The sewage should, for the most part, be collected by a system of intercepting sewers running along the waterfront and extending to the central disposal station.

The minimum requirement in the treatment of the sewage at the main station should be the removal of the suspended solids by screens and settling basins.

The arms of the Chicago River which have no present or future value for navigation, and which are now in a foul and stagnant condition, should be filled in.

All considerable flows of especially foul liquids, such as those from the stock yards, should receive special treatment before they are discharged into the Chicago River or the drainage channel.

The disposal of the sewage in the Calumet territory is a local problem which can and should be solved by the construction of one or more purification plants.

New Grading Rules for Yellow-Pine Timber

Adopted by Board of Directors, Yellow Pine Manufacturers' Association, for Mill-Constructed Buildings

THE board of directors of the Yellow Pine Manufacturers' Association has adopted a new classification for yellow-pine timber for mill construction which differs from all previous classifications adopted by associations, in that the grading is based on strength quality and not solely on defects. The classification will come before the convention of the association for adoption in July, and if past experience is any criterion the new rules will probably be accepted, since they now bear the approval of the board of directors. The new classification was worked out by the committee on classification of structural materials, composed of M. B. Nelson, chairman; I. H. Fetty, C. E. Slagle, W. J. Haynen, J. H. Eddy and J. W. Martin. The committee was assisted by John A. Newlin, engineer in charge of tests, U. S. Forest Products Laboratory, Madison, Wis.; O. T. Swan, in charge of Office of Industrial Investigations, U. S. Forest Service, Washington, D. C., and A. T. North, consulting engineer of the association.

New Classification

The new classification follows:

The classification of structural material is a question of strength and durability.

In untreated material this can best be determined by its density, as represented by the proportion of summerwood to springwood in the annual-growth rings.

Growth rings are composed of bands of hard, dark, resinous summerwood, and lighter, softer, less resinous springwood in each growth ring, and the greater the proportion of summerwood the greater the strength and durability of the timber.

Dense wood shows on cross-section an average of not less than eight growth rings per inch, measured over the third, fourth and fifth inches, on a radial line from pith to circumference, containing in the greater number of the rings one-quarter or more of summerwood, or may have an average of six or seven rings as above, provided in the greater number of rings one-third or more of the ring is summerwood, or wider ringed material if in the greater number of rings one-half or more of the ring is summerwood as above, and must show a sharp contrast in color between springwood and summerwood.

Defects

The weakening effect of knots is in proportion to the distortion from a straight line of the grain of the wood and to the degree in which they are ingrown with the adjoining and connecting growth grains; therefore, sound knots only half an inch in diameter or less may be disregarded in considering either strength or durability.

The location of defects in all stringer forms of beams is of vital importance; therefore, beams whose depths exceed their thickness by 3 in. or more should be divided for classification purposes by imaginary lines into three volumes, namely: Volume 1 consists of the lower one-fourth of the depth in the middle one-half of the length; volume 2 consists of the upper one-fourth of the depth in the middle one-half of the length; volume 3 consists of the remainder of the beam or stick.

Sapwood is as strong as heartwood, but not so durable, and, therefore, the percentage or proportion of heartwood desired should be expressed in any contract.

In a classification it would be cumbersome to attempt to enumerate all defects; therefore, defects allowed, whether specified or not, are understood to be equivalent in damaging effect to those mentioned applying to the material under consideration.

Structural material shall be classified as select structural and as No. 1 structural—sizes 6 x 8 in. and up for timbers and 3 x 12 in. and up for joists.

Standard Sizes for Rough Timbers.—Rough timber sawn to standard size shall mean that they shall not be more than ¼ in. scant of count size; for example, a 12 x 12-in. timber rough shall measure not less than 11¾ x 11¾ in.

Standard Dressing on Rough Timbers.—Standard dressing shall mean that ¼ in. shall be allowed for dressing each surface; for example, a 12 x 12-in. timber after dressing on four sides shall measure 11½ x 11½ in.

Grades

Select Structural.—All timber shall be sound and sawed to standard sizes, dense, free from such defects as ringshake showing on the faces, injurious cross-grain, unsound knots, and decay.

Stringer forms must not have incased or large sound knots in volume 1, must not have large incased knots in volume 2 or unsound knots in volume 3; beam, post, sill and other forms may have sound knots or hard, firm incased knots the aggregate diameter of which does not exceed the width of the face they are in, but no one knot shall exceed 4 in. in diameter; stringer forms shall show three-quarters heart at any point on the narrow faces, and post, beam, sill and other forms more nearly square shall show three-quarters heart on all faces at any point.

The measure of knots shall be at right angles with the grain of the knot.

No. 1 Structural.—No. 1 structural shall include timber answering in all respects to select structural, except that a greater proportion of sap or no restriction as to sap will be allowed, making timbers suitable for treatment and distinguishing them from No. 1 common timber.

BERLIN'S WATER CONSUMPTION for the year ended March 31, 1913, was about 19,500,000,000 U. S. gal., according to a review of the municipal report in "The Surveyor and Municipal and County Engineer." The average per capita daily consumption was 24.08 U. S. gal. The maximum, which occurred on July 13, was 35.76 gal. and the minimum, on April 8, 16.08 gal. The proportion of minimum, average and maximum was, therefore, approximately 4:6:9, or 1:1½:2¼.

Ridge Block Construction in Filters at Evanston

Details of Construction and Cost of Monolithic, Molded-in-Place Strainer System

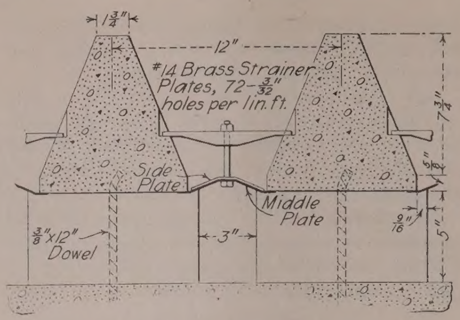
By CHESTER GORDON GILLESPIE
Resident Engineer, Evanston, Illinois

MOLDED-IN-PLACE, monolithic ridge blocks were placed in the filter strainer system at Evanston, Ill. The strainer system conforms in general to that described in the Engineering Record of Nov. 22, 1913, page 578, but a few details have been modified to eliminate prospective difficulties which have appeared at other plants. In order to reduce, during washes, the excessive velocity pressure on the strainer plates over the risers feeding the central channel, a ⅛-in. brass deflector plate 2 in. smaller in diameter than the bellmouth of the riser pipe was anchored to the bellmouth casting by four ⅜-in. brass bolts and stay nuts so that the plate is held 3 in. above the riser. Other modifications comprise strengthening the concrete edge on which the middle plate bears by casting a ⅝-in. vertical face on the bridge block; widening the ledge on which the side plates bear to 9/16 in., although in construction this varies from ⅜ to ¾ in.; using a heavier, No. 14, and more ductile strainer plate; replacing the holding-down screen by increasing the gravel layers from 9 to 12 in.

Drilling for Dowels

Monolithic construction of the filter boxes required a concentrated form bracing near the floor surface, preventing a careful screeding and interfering with the placing of the holding-down dowels for the ridge blocks in the green concrete. After the concrete lateral gutters for wash-water were constructed, dowel holes were drilled in the floor and rods with a slight hook upset at each end were grouted in.

Tests of the holding-down power of the grouting by a lever indicated that 16 per cent of the dowels pulled out before the working stress came on them. In every case failure was due to pockets of unremoved inert drilling dust which formed a pasty mass adhering to the rod, or to an inverted bellmouth at the entrance to the hole, which reduced the effective depth. Later a steam jet with side holes was used effectively to clean out the holes. Deeper holes and longer rods remedied the first trouble from flaring holes. Occasionally when it was desired to pour the ridge blocks before the grout had set long enough to warrant a test, the dowels were wedged with steel wedges. Immediate tests showed a holding-down power of several times the working requirement.



Ridge Blocks and Strainer Plates

Drilling Costs

By hand holes were drilled with a ⅞-in. diamond drill at a cost of 17 cents per hole. A small rotary electric drill showed a greater cost. With a 40-lb. Ingersoll-Rand steam jack drill, using a 1⅝-in. star drill, operated by a $5 driller, the cost was 2.8 cents per hole, not including charge for steam or depreciation. Holes were cleaned and dowels grouted and tested at a cost of 4.8 cents each.

Ridge Block Forms

Forms for one-half of a filter were made in the shop but for speed work was done by one-quarter filters. Forms were coated with a heated mixture of paraffine cut with kerosene after each use. Lugs for the recesses were kept well painted with a mixture of oil, lamp black and graphite, a procedure that prevented sticking during removal. Each form had a probable life of about 50 uses, as they were all in good condition at the close of work after 24 uses.

Roughness of the floor decreased the speed of form setting about 15 per cent, much pointing with a weak lime and cement mortar at depressions beneath forms being required.

Concrete was mixed wet in the proportions of 1 part Marquette Portland cement, 1¾ parts sand and ¼ part stone in a small Smith hand mixer on the filter roof and lowered through the manholes by rope and buckets. As most of the work was done in the winter it was necessary to lay homemade heating coils over the concrete during the setting period. Forms even in the coldest weather could be removed, leaving clean surfaces within three days. Ordinarily two days was sufficient time.

Ridge Block Costs

Details of the ridge block costs per cubic yard and cleaning out the channels ready for the strainers are:

Costs per Cubic Yard of Concrete Ridge Blocks

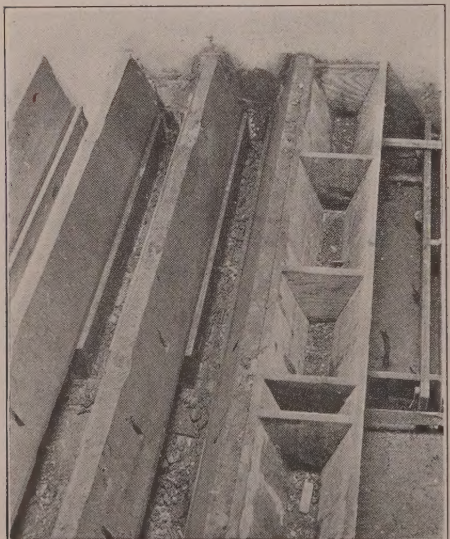
Item	Material	Foreman	Laborers	Carpenters	Total
Forms — Lumber	*$0.346	.....	.....	.....	$0.346
First or shop cost	.....	$0.060	$0.275	$1.118	1.453
Setting	.....	0.345	0.184	8.110	8.639
Wrecking and cleaning	.....	0.051	1.288	.....	1.339
Pointing	.....	.....	0.551	.....	0.551
Total for forms	$0.346	$0.456	$2.298	$9.228	$12.328
Steel—Stock	*0.908	.....	.....	.....	0.908
Placing	.....	.....	0.321	.....	0.321
Concrete—Stock	*5.700	.....	.....	.....	5.700
Mixing and placing	.....	0.526	3.150	.....	3.676
Heating	*0.388	0.143	0.712	.....	1.243
Cleaning	.....	.....	*2.700	.....	2.700
Grand total	$7.342	$1.125	$9.181	$9.228	$26.876

*Estimated.

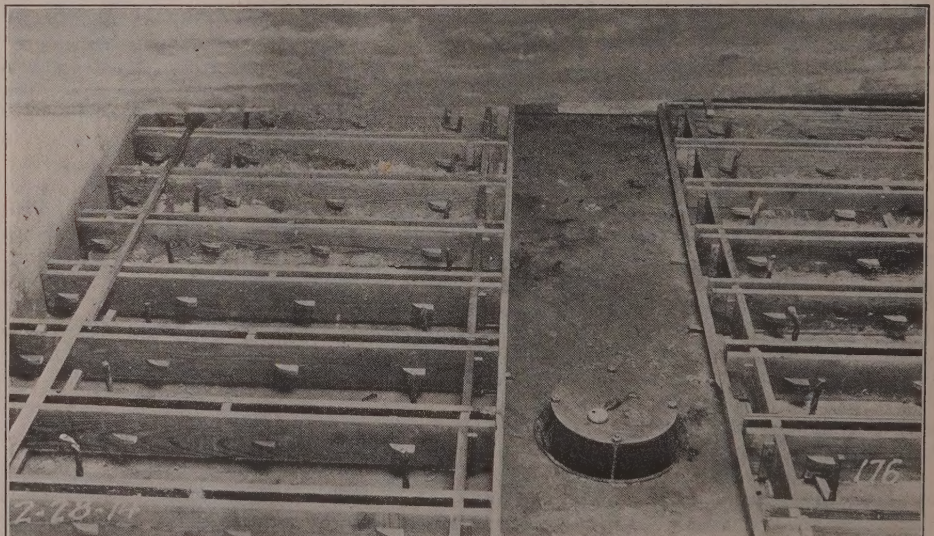
Rates per day of eight hours paid were: Carpenter foreman, $6; labor foreman, $5; carpenter, $5.20; labor, $3.20. In terms of sand surface the cost is 53 cents per square foot.

The work was done under the direction of F. M. Gallamore, superintendent for the sub-contractor, the Macdonald Engineering Company, of Chicago. The Norwood Engineering Company, general contractor, placed the strainer plates. George W. Fuller and Langdon Pearse were the designing and constructing engineers.

DIAMOND DRILL HOLES for blasting have been bored to an average depth of 63 ft. in the quarry of the Hercules Sandstone Company at Tenino, Wash. A Sullivan Class C diamond-bore drill machine with A fittings bored 1 13/16-in. holes about 9 ft. apart and 30 ft. back from the 70-ft. face. The holes were grooved in the plane of the row, sprung with dynamite, and loaded to within 10 ft. of the surface with black powder. The cost of drilling and grooving was about 28 cents per linear foot, and one blast of sixteen holes threw about 40,000 tons of rock at a drilling cost of $7 per 1000 tons.



Ridge Blocks Ready for Plates



Deflector Used over Riser Pipe

Analysis of Trenching Methods and Costs

Comparative Data on Sandy Soil Excavations for a Model Town by Hand Methods and by the Use of a Steam Shovel

By RICHARD T. DANA
Consulting Engineer, New York City

STUDIES were made some time ago under the writer's direction to determine whether under certain stringent conditions it would be economical to construct small trenches in sandy soil with a specially designed steam shovel, and in any event to determine the economic factors that enter into such work. The observations were made by experienced inspectors who had no interest in the results of their work except to see that they correctly represented the actual facts. The work was done during the summer of 1910, within the corporate limits of New York City, with ordinary, average labor and under average conditions. The operations took place in the early stages of a large land development of the "model town" order, and at the time when the observations were made the work in general was proceeding with about average efficiency. The men at that time were receiving no bonus and the foremen had not developed the more than ordinary ability in handling their work that came a month or two later. These data, therefore, are fairly representative of normal conditions on average work of this class. The field data and time studies were made by Allan C. Haskell, to whom my sincere acknowledgments are due for the excellence of the work.

At the inception of the work it was claimed by certain parties that a light steam shovel would be economical in the trenches, although it was recognized that the sandy nature of the soil and the large preponderance of curves in the trench lines militated strongly against this idea. The plan, however, was given a thorough test with the conclusive results shown below.

The trenches were dug for the purpose of laying sanitary and storm-water sewer pipe of varying diameters from 4 to 36 in. of tile and concrete, and also for cast-iron water pipe from 4 to 12 in. in size. Nearly all of the work was on curves, since the sewer and pipe lines had to be between the curbs of the streets, and curved streets, while not economical to construct, were considered highly artistic and, therefore, desirable notwithstanding the extra cost.

The shovel was of the 25-ton, revolving type, mounted on traction wheels and capable of turning around a complete circle. The entire mechanism was under the control of the runner, and the shovel was fitted with a dipper arm and 1-yd. dipper specially designed for trench work, with an interchangeable boom capable of handling a small orange-peel bucket for cellar or stock-pile work. The platform upon which the shovel worked consisted of twelve timbers of 12 x 12-in. section, spaced 4 in. apart, and bolted together in sections of three. These twelve timbers rested upon planks laid upon the ground. On top of the timbers and running transversely to them, near each end, were planks upon which the traction wheels rested directly. To move up after a completed section had been sheeted, the shovel swung around and with a chain picked up one of the sections, and swung back again and placed it upon the planks which the laborers had laid ahead. Then after the plank track had been laid, the shovel moved forward under its own power.

STEAM-SHOVEL TRENCHING

In Table 1 the time given under "time worked by shovel" includes all time of moving up and waiting for sheeters before moving up, as well as the actual digging time of shovel. It is that time which could rightly be charged to the shovel. But when the sheeting gave out and the shovel was idle, as frequently happened during the experiment, such time was not charged to the shovel. This suggests one very important point, namely, always to have sufficient supplies and material on hand to prevent high-priced machinery from being idle. On two days (Table 3, observations 10 and 11) the shovel was trying to work in sand, but it made a very poor showing, for the sand kept caving in from the sides of the trench. The caving finally became so bad that for fear the whole machine would fall into the trench it was moved to a new cut. The material was much firmer here and the unit costs would have been much lower if an unexpected difficulty had not presented itself. A curve was reached and the shovel was unable to negotiate it. On one day 14 per cent of the time was thus lost, and on another 50 per cent. The best showing was made when the banks held up well and where no curves were encountered, but even here the cost per cubic yard was about 15 cents.

In order that a clear idea may be had of the shovel performance, one day's work will be briefly summarized and analyzed. (See Table 3, observation 7.) The costs in Table 1 do not include superintendence, preparatory costs (pro rata cost of unloading and moving the shovel to the first cut), overhead charges or profit.

In justice to the steam shovel as the contractor's friend, it should be stated that other trench work, under less onerous conditions, has been very economically done with steam shovels. By way of compari-

TABLE 1—WORK OF 25-TON REVOLVING SHOVEL—ONE-DAY PERFORMANCE

Kind of shovel	25-ton
Capacity of dipper	1 yd.
Length of move	4 ft.
Number of moves	20
Average time to sheet trench before moving up	9.2 min.
Average time to move up	4.5 min.
Time worked by shovel	565 min.
Cut	9 ft.
Width	36 in.

Material, clay and gravel that held up well.
Remarks: Shovel idle while trench was being sheeted. Curve also caused idleness.

Performance	80 cu. yd.
Unit, cost, cubic yard	22.6 cents

Daily Cost of Operation

1 Runner	$5.00
1 Fireman	2.31
1 Laborer	1.75
1 Laborer	1.65
Supplies	4.50
Interest and depreciation, 17½ per cent on $4,500 (approx.), based on 200 working days per year	4.00
Total	$19.21

$19.21 × 565/600 = $18.10; $18.10/80 = 22.6 cents per cubic yard

Process Analysis

	Time, minutes	Per cent	Cost, cents per cubic yard
Actual digging	202	35.8	8.1
Delays: A—Sheeting trench before moving up	184	32.6	7.35
B—Moving up	90	15.9	3.6
C—Delay due to curve	89	15.7	3.55
	565	100.0	22.60

TABLE 2—DATA ON 70-TON SHOVEL AND PROCESS ANALYSIS OF ONE-DAY PERFORMANCE

Kind of shovel	70-ton
Capacity of dipper	2 yd.
Average length of move	15 ft.
Number of moves	4
Average time to move up	33¾ min.
Working time	602 min.
Cut	26 ft.
Width	16 ft.

Material: first 10 ft. top soil; next 16 ft., glacial drift

Performance	569 cu. yd.
Unit cost, cubic yard	6.7 cents

Cost of Operation

1 Runner	$5.00
1 Craneman	3.60
1 Fireman	2.00
7 Rollermen	10.50
Supplies	9.00
Interest and depreciation at 17½ per cent on $9,000 (approx.), based on 200 working days	8.00
Total	$38.10

Unit cost per cu. yd. = 38.10/569 = 6.7 cents

Process Analysis

	Time, minutes	Per cent	Cost, cents per cubic yard
Actual digging	270.5	44.9	3.01
Delays: A—Waiting on sheeters	50	8.4	0.56
B—Moving up	135	22.4	1.51
C—Waiting on cars	139.5	23.2	1.55
D—Miscellaneous	7	1.1	0.07
Total	602	100.0	6.70

Views of a Large and Small Steam Shovel Engaged in Trench Work

TABLE 3—DIGGING TRENCH BY STEAM SHOVEL—25-TON VULCAN—UNIT COSTS AND OTHER DATA

Observation	Unit cost, cents		Cut		Width, in	Equiv. 10-hr. performance		*Time Worked by shovel		Remarks
	Cu. yd.	Lin. ft.	ft.	in.		Cu. yd.	Lin. ft.	Min.	Per cent total	
1	13.1	8.5	6	10	31	147	225	600	100	Good going, no caving
2	14.9	9.0	6	6	30	129	214	390	65	Good going, no caving
3	15.6	11.2	6	6	36	126	175	515	86	Of the 86 per cent, repairs to crane engine took 15.8
4	15.6	13.0	7	6	36	123	147.5	600	100	Excellent going, straight trench and solid bank
5	16.2	17.1	9	6	36	119	113	320	53	The 47 per cent idle time waiting for sheeting
6	19.2	19.2	9		36	100	100	180	30	The 70 per cent idle time due to sheeting giving out, gravel and clay holding well
7	22.6	22.6	9		36	85	85	565	94	Inability of shovel to make a curve took 14 per cent of this 94 per cent; bank clayey and fair digging.
8	24	23.2	8	8	36	80	83	600	100	Good going with curve caused a slow up
9	36.6	38.0	9	6	36	52.5	49.6	600	100	About 50 per cent of this 100 per cent was lost due to inability of shovel to make a curve
10	39	34.8	8		36	49	55	382	63	Idle 37 per cent of time waiting for sheeting; sand caving badly
11	42.0	30.4	6	6	36	45.7	63.2	180	30	Idle 70 per cent of time due to bad cave-in, necessitating moving of shovel to a new cut
Average	23.5	20.9	8	0	36	96	108	448	74.7	

Above unit costs are based upon the shovel charges in Table 1.

*This is the time chargeable to the shovel, unit costs based on these figures.

TABLE 4—DEDUCTION OF UNIT COSTS

Average number of men	19
Yardage	163
Lineal feet of trench at 4x5 ft. 8 in	195
Material, clay and sand.	
Cost, 19 men at $1.65	$31.40
½ foreman at $3.50	1.75
¾ water boy at $1	.75
	$33.90
Per cubic yard	$33.90/163 = 20.8 cents
Per linear foot	$33.90/195 = 17.4 cents

The charge to excavation for foreman varies from 1/2 to 1/3 according to work he supervised.

TABLE 5—DIGGING TRENCH BY HAND, SHOWING UNIT COSTS AND OTHER DATA; DEPTHS UP TO 6 FT.; MATERIAL HANDLED TWICE

Observation	Unit cost, cents		Cut.		Width, in.	No. of men	Equiv. 10-hr. performance		Size of pipe	Remarks
	Cu. yd.	Lin. ft.	ft.	in.			Cu. yd.	Lin. ft.		
1	15.6	9.0	5	2	36	8	95.6	166	12-in. vit. drain	Sand, clay, few boulders
2	17.0	9.5	6	0	30	8.5	94	169	12-in. drain	Sandy material
3	19.0	7.4	5		24	10	113	292	4-in. water pipe	Hard packed sand requiring picking
4	19.4	10.6	5	8	32	11	110	200	12-in. drain	Easy digging through recent clay fill
5	20.8	17.4	5	8	48	19	163	195	24-in. drain	Easy digging earth and clay, wide trench
6	20.8	13.5	5	6	36	5.7	53.4	82	12-in. drain	Clay on top, sand beneath
7	24.0	13.1	5	6	32	14	105	193	12-in. drain	Easy digging
8	24.7	16.4	6	0	36	6	45.5	68.5	24-in. drain	Sand and clay
9	26.8	6.9	3	6	24	11	78	300	12-in. water pipe	Sand and clay
Average	20.9	11.4*	5	4	33	10.3	95.3	*175		

*Deduced from average cost per cubic yard and average 10-hour performance respectively, using average cut and width of trench.

TABLE 6—DIGGING TRENCH BY HAND, SHOWING UNIT COSTS AND OTHER DATA; DEPTHS BETWEEN 6 AND 12 FT.; MATERIAL HANDLED THREE TIMES

Observation	Unit cost, cents		Cut,		Width, in.	No. of men	Equiv. 10-hr. performance		Size of pipe	Remarks
	Cu. yd.	Lin. ft.	ft.	in.			Cu. yd.	Lin. ft.		
1	15.3	12.8	8	5	32	13.0	153.0	183.0	8-in. sewer	Easy digging, good foreman and gang
2	18.2	10.6	6	3	30	6.5	68.7	118.0		Hard top soil requiring picking, then sand and boulders
3	18.2	11.8	7	0	30	12.8	132	204	12-in. drain	Material very sandy, requiring sheeting
4	18.8	12.3	7	0	30	13	123.5	190		Hard top soil, then easy sand; some boulders
5	19.5	19.5	9	0	36	16.4	161.5	161.5	12-in. sewer	Top soil with roots, hard clay; sand; picking
6	20.0	23.3	10	6	36	9	82	70.4	12-in. sewer	Good digging, little sheeting needed
7	20.9	19.6	9	6	32	13	111.5	119	8-in. sewer	Good digging, few rocks
8	21.1	15.2	6	6	36	11	93	129	24-in. drain	Gravel and clay, latter predominating
9	21.6	19.6	7	0	42	3.2	30.4	33.6	24-in. drain	Hard packed sand; picking
10	21.8	17.0	7	0	36	14	111.5	143		Sand and clay
11	23.0	17.9	7	0	36	13	100.5	129	24-in. drain	Hard packed gravel; picking; good going
12	23.3	19.7	6	6	42	6	51.5	61	24-in. drain	Hard packed sand; picking
13	23.3	17.2	8	0	30	20	150	203	12-in. sewer	Frequent cave-ins, causing rehandling
14	24.0	18.7	7	0	36	8	60.6	78	12-in. sewer	Loam with roots first 12 to 18 in., then gravel and clay
15	24.2	29.5	11	0	36	10	75	61.5		Sand and clay
16	25.3	30.9	11	0	36	13	90.8	74.4		
17	25.6	27.3	8	3	42	9	63.5	59.5	24-in. drain	Good organization; interest in work
18	26.2	17.0	7	0	30	12	83	128		
19	27.6	23.1	9	0	30	10.4	69	82.5	8-in. sewer	Sand and clay
20	28.0	26.5	8	6	36	19	118	125	12-in. sewer	Clay and sand; hard surface; gravel
21	28.6	14.3	7	0	24	18.7	97	187	8-in. sewer	Hard digging and picking
22	29.2	28.0	10	4	30	16	97	101	8-in. sewer	Hard crust, then sand
23	29.5	24.5	7	6	30	13	81.5	98	8-in. sewer	
24	29.8	35.3	8	0	48	16	59.6	50.4	24-in. drain	Hard sand; picking
25	30.4	24.0	8	0	32	12.7	83.5	105.6	8-in. sewer	Hard picking; many small boulders up to 8 in. diameter
26	32.9	25.7	7	11	32	12.9	72	92.3	8-in. sewer	Considerable picking; close sheeting; quicksand
27	33.6	33.4	10	1	32	11.3	61.6	62	8-in. sewer	Many field stones removed by hand
28	33.8	36.4	9	9	36	12	62.5	58	24-in. drain	Little new trench opened
Average	24.8	21.8*	8	3	34.4	12.3	90.8	103.5*		

*Deduced from average cost per cubic yard and average 10-hour performance in cubic yards respectively, using the average cut and width of trench.

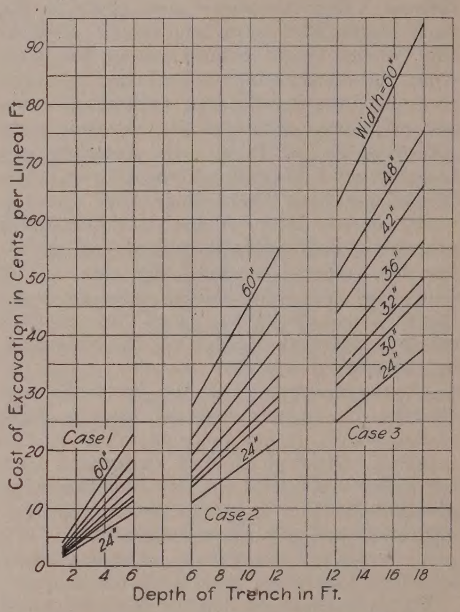
son is given below the analysis of some trench work in Chicago that cost 6.7 cents per cubic yard (direct expense), a description of which was published by the Bucyrus Company in "Hand Book of Steam Shovel Work."

The material was top soil to a depth of 10 ft., then glacial drift to the bottom of the trench, which had a total depth of 26 ft. The width of cut was 16 ft.; the line was practically straight. The shovel was a 70-ton Bucyrus, removed from its trucks and placed upon a 38 x 24-ft. platform, mounted on wooden rollers. To move the shovel forward or backward a cable with block and fall, attached to a deadman, was wound up by the shovel engines. The excavation was made in two benches, the first 10 ft. and the second 16 ft. This was in order to get fine stuff for backfilling. Sheeting was necessary only to a depth of 10 ft. Trimmers followed the shovel and prepared the sides of the trench for the sheeters. When the shovel had finished a section of the first 10-ft. bench it moved back and started the second. On this second bench the shovel worked between two I-beams, braced every 20 ft. by jacks. These I-beams held the sheeting in position until the shovel had passed, when the regular blocking was replaced. The I-beams were so attached to the shovel that when it moved forward and the jacks were slightly loosened they could be dragged along into their new position. Spoil from the trench was loaded into trains of dump cars and hauled by dinkeys to the dump.

As in the case with which this is compared, no account is taken of overhead charges, etc.

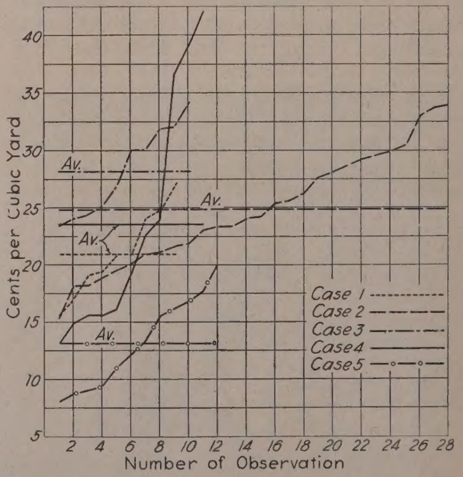
From Table 2 it will be seen that the total cost in the last case, with favorable conditions, was 1.4 cents less than the "actual digging" charge, and 15.9 cents less than the "total charge" of Table 1, where conditions for the use of a steam shovel were unfavorable.

In general the material through which the trenches were dug was composed of 1 ft. of top soil, 3 ft. of hard packed clay



Costs per Lineal Foot for Digging Trench

Case 1. Material handled twice—depth of trench 0 to 6 ft.—20.9 cents.
Case 2. Material handled three times—depth 6 to 12 ft.—24.8 cents.
Case 3. Material handled four times—depth 12 to 18 ft.—28.2 cents.



Costs of Digging and Backfilling Trench

Case 1. Material handled twice, by hand—depth of trench 0 to 6 ft.
Case 2. Material handled three times, by hand—depth 6 to 12 ft.
Case 3. Material handled four times, by hand—depth 12 to 18 ft.
Case 4. Material handled by steam shovel—depth 6.6 to 9.6 ft.
Case 5. Backfilling—material handled once and tamped.

requiring picking and beneath this sand and fine gravel. As the geological character of the section is that of a glacial deposit, nests of stones, averaging 1 ft. in diameter, were frequently encountered. These were lifted out by hand, if not too large; otherwise they were removed with a chain hoist. After the trench had passed a depth of 4 ft. a man was usually stationed on the bank to cast back the earth removed by the trench diggers. At a depth of 7 ft. a staging was required, thus necessitating three handlings. A second staging was necessary when the trench became more than 12 ft. deep, causing four handlings. For slight depths bracing only was necessary, but when the sand was reached sheeting more or less close had to be driven. The last few inches of trench were dug by the pipe layer who usually cast the earth back on the pipes already laid, and the cost of this excavation was usually charged to pipe laying. In the deep trenches the last foot or more was usually excavated in this manner.

Table 5 is a summary of the records of such observations where two handlings were necessary; Table 6 of twenty-eight observations where three handlings were required; Table 7, ten observations under four handlings. Diagram 1 shows the cost per lineal foot of digging trench by hand for various depths and widths.

To show how these unit costs were deduced one example will be given in Table 4. (See observation 4, Table 5.)

The price of common labor upon which unit costs have been based ranged between 16½ and 17½ cents per hour; for foremen, between $3.50 and $4 per ten-hour day, and for waterboy, $1 per day.

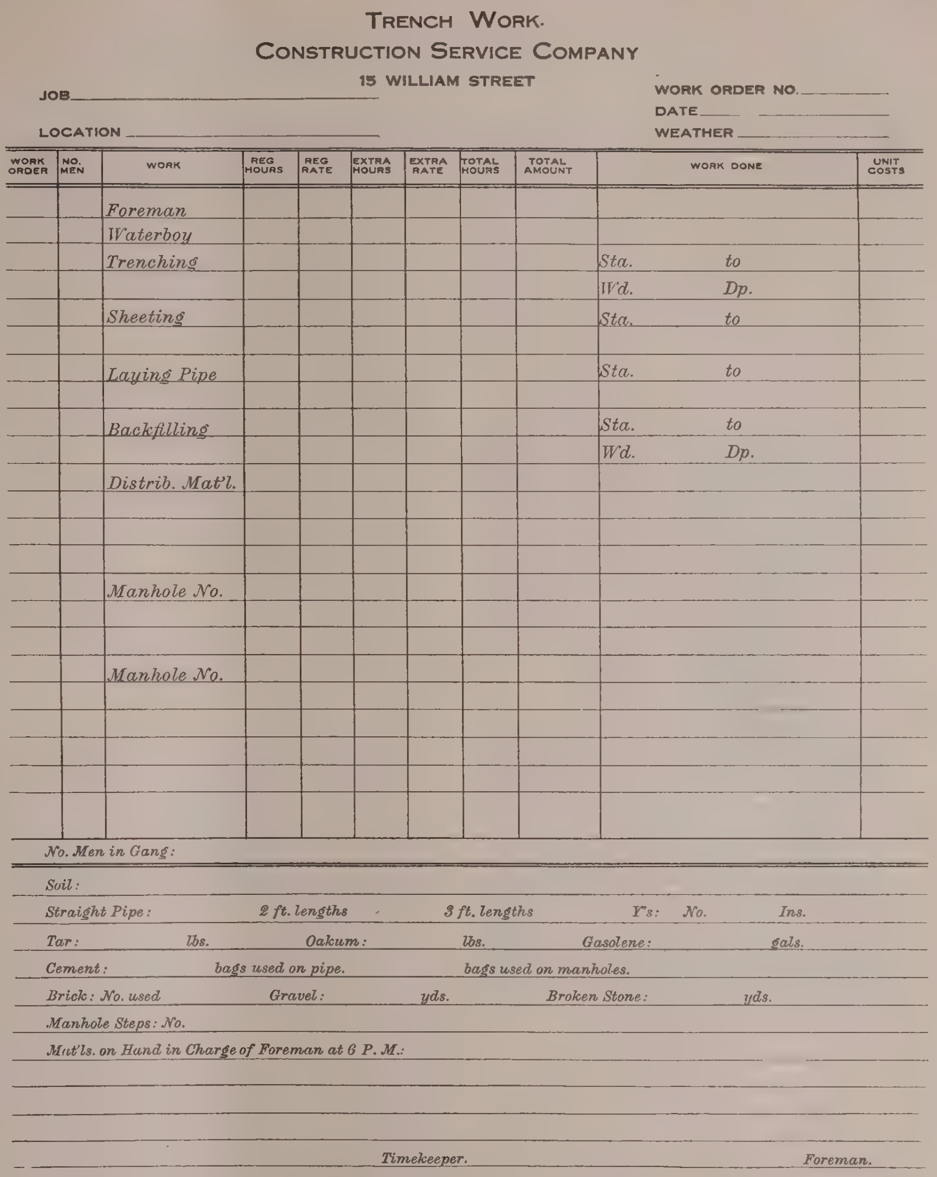
Later in the season a report form was devised and installed with success, a copy of the blank form being shown. For each particular piece of work, or job, a "work order" was issued and all items making up the cost of the job were charged against this work order number. At times men would be taken from one job to do work on another. If the amount of this work was not very large their performance was entered on the regular work order report, but the work order number in question was entered in the column at the left hand side of the sheet.

BACKFILLING AND TAMPING

The unit cost of backfilling is not directly affected by the depth of trench, but it is vastly influenced by the method and the tools employed. These methods must depend somewhat on the size of the earth pile that is to be thrown back into the trench. Generally it is better when shoveling backfill by hand for a limited number of men to stand on the original ground level, shoveling at the end of the pile and throwing to the right or left than for them to get on top of the excavated earth and dig it out from under their feet. When the trench is very deep and there is consequently a large pile of earth to be thrown back, the men necessarily work at some disadvantage. Indirectly, then, the unit cost of the backfilling is to some extent dependent upon the depth of trench. In using scrapers to backfill mules can and will approach much nearer to the edge of the trench without danger of falling into it than can horses.

It is important when studying the costs in Table 8 to remember that thorough tamping was specified and rigidly enforced. When water could be obtained sections of the trench were dammed at each end, the trench filled with water and the earth cast therein. The earth composing these dams was thoroughly tamped by hand.

Table 8 gives unit costs and other data from twelve sets of observations on backfilling and tamping. The average unit cost



TRENCH WORK.
CONSTRUCTION SERVICE COMPANY
15 WILLIAM STREET

JOB
LOCATION
WORK ORDER NO.
DATE
WEATHER

WORK ORDER	NO. MEN	WORK	REG HOURS	REG RATE	EXTRA HOURS	EXTRA RATE	TOTAL HOURS	TOTAL AMOUNT	WORK DONE	UNIT COSTS
		Foreman								
		Waterboy								
		Trenching							*Sta. to*	
									Wd. Dp.	
		Sheeting							*Sta. to*	
		Laying Pipe							*Sta. to*	
		Backfilling							*Sta. to*	
									Wd. Dp.	
		Distrib. Mat'l.								
		Manhole No.								
		Manhole No.								

No. Men in Gang:
Soil:
Straight Pipe: *2 ft. lengths* *3 ft. lengths* *Y's: No.* *Ins.*
Tar: *lbs.* *Oakum:* *lbs.* *Gasolene:* *gals.*
Cement: *bags used on pipe.* *bags used on manholes.*
Brick: No. used *Gravel:* *yds.* *Broken Stone:* *yds.*
Manhole Steps: No.
Mat'ls. on Hand in Charge of Foreman at 6 P. M.:
Timekeeper. *Foreman.*

Specimen Work Order for Trench Work

TABLE 7—DIGGING TRENCH BY HAND, SHOWING UNIT COSTS AND OTHER DATA; DEPTHS BETWEEN 12 AND 18 FT.; MATERIAL HANDLED FOUR TIMES

Observation	Unit cost, cents		Cut, ft. in.	Width, in.	No. of men	Equiv. 10-hr. performance		Size of pipe	Remarks
	Cu. yd.	Lin. ft.				Cu. yd.	Lin. ft.		
1	23.4	36.4	14	36	13	99.4	63.8	12-in. sewer	Sand and clay
2	24.0	40.2	15	36	13.5	99.5	59.5	12-in. sewer	Sand and clay
3	24.4	43.3	16	36	13	56	31.5	12-in. sewer	Excellent organization; top soil, then clay and gravel
4	25.1	44.6	16	36	16	112	63	12-in. sewer	Excellent organization; top soil, then clay and gravel
5	26.9	47.9	16	36	13	89	50	12-in. sewer	Top loam with roots, then gravel and caving sand
6	30.0	53.4	16	36	16	93.4	52.5	12-in. sewer	Excellent organization; gravel; picking
7	30.0	56.5	17	36	17	103	54.7	12-in. sewer	Difficult, bad caving; double rows of jacks
8	31.8	53	15	36	15.5	91	54.5	12-in. sewer	Top loam with roots, then gravel and caving sand
9	32.0	42.6	12	36	16.5	90	67.5	24-in. drain	Sand with bad caving tendency
10	34.2	44.9	11.10	36	15.0	80	61	24-in. drain	Sand with bad caving tendency
Average	28.2	*46.8	14.11	36	14.3	91.3	*55		

*Deduced from average cost per cubic yard and average ten-hour performance respectively, using the average cut and width. Included in the above unit costs is a proportion of the charges for foreman and water boy.

TABLE 8—BACK FILLING, SHOWING UNIT COSTS AND OTHER DATA

Observation	Unit cost, cts. per cu. yd.	Equiv. 10-hr. performance, cu. yd.	Average number of men	Remarks
1	8.05	113.00	5½	Tamped by hand
2	8.75	61.50	3	Tamped by hand
3	9.05	76.6	3	Tamped by hand
4	9.35	153	7	Tamped by hand
5	11.00	82.5	3	Tamped by hand
6	12.2	130	10	Tamped by hand
7	13.3	85	6	Tamped by hand
8	15.5	92	7	Tamped by hand
9	16.2	59.5	4	Partly by hand, partly using scoop and mules, and partly using water
10	16.8	44.4	4	Tamped by hand
11	17.7	61.1	6	Tamped by hand
12	20.0	53	6	Tamped by hand
Average	13.16	81.8	5.4	
A	7.65	4.54	19	Water used exclusively

In the above unit costs are included proportionate charges for foreman and water boy.

was 13 cents. In this table one case is recorded where water was used exclusively. The unit cost was 7.6 cents, about one-half of what it was without water. The cost of water is not included in this 7.6 cents. Observation 9 is a case where part of the work was done by hand with tamping, partly by hand with water and partly by a drag scraper or scoop, pulled by mules and dumping the earth into the water. One scheme tried was to fill in two sections of trench by hand to make roadways for the mules and then to haul the loaded scoop across one of these and dump it. Another was to keep the mules on one side of the trench and the scoop on the other and connect them by a chain. The fact that the mules were green and timid in the first case, and the necessity of pulling the empty scoop back by hand in the second, were the causes of failure. With a pair of steady horses the first scheme would undoubtedly have been successful.

Sheeting in Trench under Small Shovel

Cost of Drainage Excavation

Excavation of 60,336 cu. yd. on the Stony Creek diversion of the Sanitary District of Chicago was done last fall at a unit cost of 51.9 cents per yard. This was considered to compare favorably with other work for the Sanitary District, especially as the cross-section of the channel was small, increasing the cost of moving the steam shovel; as the plant charge was large in proportion to the yardage; as it was necessary to construct a dump trestle and haul the material half a mile, and as the price of common labor had advanced from 17½ to 25 cents per hour. The cost items include $8,325.71 for plant cost and installation, $24,906.06 for operation and $5,628.38 for bridges and miscellaneous work. The plant installed was old and required an unusual amount of repairs. An analysis of items shows that the total cost per yard was made up of 18 cents for steam-shovel work, 13.7 cents for train work, 19.3 cents for track and dump work, and 0.9 cent for pumping. On the Sag Harbor feeder improvement 29,355 cu. yd. were dredged in 1912 at a cost of 8.25 cents per yard for plant and 14.95 cents for operation—a total of 23.2 cents per yard.

Seepage in Relation to Irrigation Project Drainage

Proposal to Pump Silt, Loosened by Giants, into Canal in Order to Form Lining and Reduce Losses

In the Engineering Record of April 25, 1914, page 468, was a description of the drainage system of the North Side Minidoka irrigation project of the U. S. Reclamation Service, taken from a paper read by F. N. Cronholm before the Idaho Society of Engineers. In that paper suggestions were made as to remedial measures, other than the drainage system being installed. These consisted of more intelligent application of the water to the land and the lining of the canals.

Now that the Government has taken over the distribution of water from laterals, and is enlarging them, it is hoped, Mr. Cronholm said, that the rotation of water as well as the use of large heads will come into vogue, because with such practice lesser loss will result and the crop production will be greater.

Canal Lining

Canal lining would probably result in great economy, he continued, in securing an adequate drainage system, for on the assumption that lateral losses equal the canal losses, there is a total loss on this project during the irrigation season of 192,000 acre-feet. If one-half of this amount can be saved by silting the canals and laterals, 228 sec.-ft. will be prevented from entering the soil during the irrigation season.

Another illustration showing the economy of canal lining in places where such losses contribute towards the raising of the ground-water plane is a case in the northeast part of the project where the main canal begins to branch. At this particular place the losses are from two to three times greater than the general average per square foot of wetted area. In this stretch of 4000 lineal feet of canal, about 17.5 sec.-ft. are lost throughout the irrigation season—the rate of loss being 4.28 ft. in depth per day per square foot of wetted area. To construct a drain which would remove this amount of water would necessitate an expenditure of not less than $40,000, whereas if the canal is lined a natural saving would result, as it is reasonable to assume that ultimately this 17.5 sec.-ft. of water must be removed by way of drains, because it is not at all unlikely that the ideal underground drainage which seemingly exists may become clogged.

No canal lining has as yet been undertaken, but it is very probable, especially as satisfactory material for silting has been found along the canal near the headworks. Should this be undertaken, giants will be employed in mining and breaking the material. The water from the giants will carry the material in suspension to a centrifugal pump, which in turn will deliver the material to the canal. As the material lies below gravity, water additional to that furnished the giants will be diverted from the main canal in order to work the pump to full capacity. As a basis for an estimate, it was computed that by covering the wetted area of all canals and laterals with 1 in. of silt, 50,000 cu. yd. of material would be needed and that, in order to keep from over-silting or from depositing too much silt in one place, it would require two months to do the lining. In placing 50,000 cu. yd. in two months in a canal carrying 1100 sec.-ft. a mixture of 0.04 of 1 per cent of solids would be obtained in the canal, and this, it is estimated, would practically fill all voids and thereby greatly reduce the seepage losses in canals. While it is more or less of an assumption as to ultimate results and methods in doing the silting, the fact remains, Mr. Cronholm stated, that if experiments substantiate our belief not only will the seepage from canals and laterals diminish, but the silt which is carried onto the land will effect equally as good results.

Pneumatic Placing of Concrete

Internal Wear on Pipe Lines and Methods of Efficient Air Propulsion

The pneumatic placing of concrete puts high requirements on the conveying pipe line. This has compelled the companies specializing in such work to determine by experiment what kind of pipe gives the lowest cost per cubic yard of concrete placed, irrespective of its first cost. The Pneumatic Concrete Placing Company, of New York City, which uses an 8-in. pipe line for conveying concrete, has made extensive experiments on internal erosion of pipe. At first spiral riveted pipe was used, but it was found that the rivet heads were worn off and that the pipe consequently went to pieces after about 500 cu. yd. had passed through it. The cost of pipe, including flanges and accessories, was found to be 12 cents per cubic yard of concrete placed.

As light pipe is desirable, owing to the continuous shifting, lock-joint pipe was used next. This pipe is similar to the spiral riveted tubes except that the riveted joint is replaced by a pressed lap joint. This pipe acted very much the same as the spiral riveted pipe; it was worn out after having passed 500 cu. yd. and the cost was 12 cents per cubic yard of concrete placed.

English Seamless Tubing

The company then tried English seamless steam tubing. The first cost of this pipe is high, but it was found that 10,000 cu. yd. of concrete could be conveyed before the pipe was worn out and that the cost of pipe per cubic yard placed was only 4 cents. For pipe lines that are to remain in place a week or more, the company uses oil-well casing, which will pass 18,000 cu. yd. before it is worn out and the cost of which is only 2½ cents per cubic yard placed. The hardest wear in the pipe line is at points of deflection, and it has been found that a 90-deg. cast-iron elbow will be worn out after about 2000 cu. yd. of concrete had passed through it.

The company has also had another experience with its pipe lines. When concrete had to be elevated the pipe was placed on a slope, but it was found that the air passed over the concrete. Vertical risers were, therefore, experimented with and corrected the trouble. When concrete is conveyed through long pipes the same trouble is experienced. The batch flattens out in the pipe permitting the air to pass over it. To collect the batch again so that it will form a plug, vertical risers are inserted at intervals. This is accomplished by inserting a 90-deg. elbow, a piece of vertical pipe, two 90-deg. elbows, another vertical pipe and a 90-deg. elbow. The arrangement resembles expansion curves on steam pipe lines.

Convention of the American Waterworks Association

Abstracts of Some of the Technical Papers Presented at the
Thirty-Fourth Annual Meeting in Philadelphia, Held May 11-15

Liquid-Chlorine Disinfection at Philadelphia

By FRANCIS D. WEST
Chemist in Charge, Torresdale Laboratory

PERMANENT apparatus for the disinfection of water by liquid chlorine was installed for the first time in Philadelphia at the Queen Lane filter plant in September, 1913. This method of treatment has since superseded the use of hypochlorite at the city's five water-purification works. There are in use ten liquid-chlorine plants, two at each filter plant. The installation of the new equipment was begun at the Torresdale filters on Nov. 25, 1913. The apparatus was supplied by the Electro-Bleaching Gas Company and is of the general type described in the Engineering Record of April 12, 1913, page 417. The ten chlorinators cost $9750.

The feed at Torresdale is controlled by two regulating valves, the first reducing the initial pressure to about 15 lb. per square inch, and the second for regulating the pressure through a range sufficient to give the desired amount of gas. The gas then passes through a 3/8-in. hard-rubber pipe to the bottom of the first absorption tower, 8 ft. high, filled with coke. This tower is sealed; the excess gas passes through a second pipe to a second tower.

New Apparatus for Application of Liquid Chlorine

The accompanying table gives comparative results of treatment with bleach and with liquid chlorine. The water during the period Feb. 22 to March 31, 1912, was the worst that the Torresdale plant has been called upon to handle. Comparing test 1A with 1B, the efficiency of 1.3 lb. per million gallons compared with 9 lb. bleach, a ratio of 1 to 7, was 90 against 64 per cent. Comparing test 1C with 1A, this is a substitution of 0.8 lb. of liquid chlorine for 4.5 lb. of bleach, or a ratio of 1 to 6; the number of bacteria in the effluent was 118 against 260 and the per cent of B. coli was correspondingly lower.

There are several claims made for liquid chlorine, an analysis of which may prove of value. Liquid chlorine is an absolutely pure chemical, concentrated in small cylinders, while chloride of lime is bulky, requiring large space for storing. About ten to eleven times as much space is required for a bleach as for a liquid-chlorine plant. With efficient controlling devices liquid chlorine will eliminate the disagreeable odors and corrosive influences of chloride of lime; consequently, the installation may be placed in positions where the use of chloride of lime is impossible. Ordinarily this is true. The odor of chlorine at Torresdale is hardly noticeable, but there are times when, due to carelessness or accidents, the atmosphere has been unbearable and chlorine has escaped in large amounts. This has usually been caused by the failure of the water supply used for absorption and not by the apparatus.

Liquid chlorine will retain its full efficiency over an unlimited time, whereas chloride of lime deteriorates rapidly. This is one of the best arguments for liquid chlorine, especially for use in small installations.

No taste or odor appears in water treated with liquid chlorine. A heavy overdose can be given without complaints.

Liquid chlorine necessitates no labor cost, while chloride of lime does. This is true; but a liquid chlorine requires skilled supervision to be operated properly and is not foolproof. Liquid chlorine leaves no sludge. This is self-evident. Liquid chlorine will reduce the amount of alum needed for bacterial removal. There can be no question but that in cases where the water is comparatively clear and where alum is used chiefly for bacterial removal, if liquid chlorine is used before filtration it will make a marked saving in the cost of alum and in many cases will not only pay for itself but will decrease the general cost of the plant. The feed of liquid chlorine is regular from hour to hour, but the feed of chloride of lime varies constantly.

The chief objection to the use of liquid chlorine lies in the concentrated energy of the material itself. If liquid chlorine is set free in small inclosures it will cause nausea. With ordinary common sense and judgment on the part of the operator this is not likely to happen. The greatest danger lies in faulty cylinders and faulty valves. If the cylinder valve will not turn off or if the cylinder leaks it must be gotten out to the open air and the chlorine allowed to escape.

COMPARATIVE COSTS

The following estimated comparative figures are submitted: Chloride of lime costs us from $1.22 to $1.70 per 100 lb.; the usual quotation was $1.34 and the average figure $1.40. Taking $1.40 as a basis, we used during 1913 an average of a little more than 1200 lb. a day, or $16.80 a day for powder.

Two laborers at 25 cents per hour were employed for eight hours, or $4 per day, making a total cost of $20.80 per day exclusive of repairs, sample collecting or laboratory analyses.

The amount of liquid chlorine used April 10, 180 lb., would cost at 10 cents per pound $18 per day. We have now passed the worst periods of the year, February and March, when we used 234 lb. a day, costing $23.40. It is expected that we will be able to reduce the amount of liquid chlorine to at least 3/4 lb. per million, or 120 lb. a day.

COMPARATIVE RESULTS OF TREATMENT WITH BLEACH AND LIQUID CHLORINE AT TORRESDALE

Test	Date	Treatment per mil. gal.	Bacteria per cu. cm. Applied	Filt.	Treated	Per cent removed	B. coli in 1 cu. cm.	10 cu. cm.
1A	Feb. 22-March 31, 1912	9-lb. bleach	17,000	760	260	64	2	13
1B	Feb. 22-March 31, 1914	1.3 lb. liq. cl.	6,800	320	30	90	0	5
1C	Jan. 1-Feb. 28, 1914	4.5 lb. bleach; 0.8 lb. liq. cl.	12,000	710	118	83	0	5
2A	January, 1913	870 lb. bleach; 150 lb. liq. cl.*	13,500	900	227	75	2	5
2B	March, 1913	234 lb. liq. cl.*	7,200	305	27	91	0	3
3A	November, 1913	1620 lb. bleach; 90 lb. liq. cl.*	8,500	170	31	82	1	8
3B	March, 1914	234 lb. liq. cl.*	7,200	305	27	91	0	3
4A	April, 1913	7-lb. bleach	3,320	50	10	80	0	0
4B	April, 1914	1-lb. liq. cl.	2,070	89	11	88	0	1
5A	February, 1914	800 lb. bleach*; 1-lb. liq. cl.	7,250	460	99	78	..	4
6A	March, 1914	1.3-lb. liq. cl.	7,200	305	27	91	0	10

*Per day.

Some supervision and handling of cylinders is required. At present the work is done by a $3 a day mechanic, who also keeps the prefilters in repair. His wages are charged against the prefilters. A charge of $1 per day would be fair for this service.

The labor cost during 1913 of $4 per day at Torresdale, with its output of 180,000,000 gal., amounted to only 2.2 cents per million gallons.

On April 21 the amount of liquid chlorine used at Torresdale was reduced to ¾ lb., or a cost of $13.50 per day exclusive of a possible charge of $1 for labor. In general the cost of the two processes should be about equal; if anything, liquid chlorine should prove cheaper.

While in some instances liquid chlorine may prove more costly than chloride of lime, the regularity with which it can be applied, its more effective action on pathogenic bacteria, the small compact apparatus and the absence of the odor of chlorine around the plant recommend it as a satisfactory substitute for hypochlorite, having, as it does, all the advantages of the latter and only some of the faults.

Old Hypochlorite Tank

Substituting Agar for Gelatine in Bacterial Counts

By W. U. C. BATON
Analyst in Charge of Laboratory, Pittsburgh

THERE is practically no difference in the ease or difficulty with which the two media, agar and gelatine, are made up. The agar at 37 deg. C. has an apparent advantage in that it will give a count at the end of twenty-four hours. On the other hand, it has the disadvantage of giving many lost results due to troublesome spreaders even with the porous cover dishes. The writer finds that the loss by liquefying of the gelatine plates is practically nil, hence the results on gelatine, while they take longer, are more certain.

It has been the writer's experience that the gelatine count will frequently indicate conditions which would never be shown by the agar 37 deg. count alone. Such conditions are an abnormal fluctuation in a reservoir, breaks or other abnormal drafts on pipe lines, abnormal growths in filter underdrains, springs and wells sensitive to local rains through shallow drainage and other similar problems which many have doubtless encountered. These may not have an immediate direct bearing on the public health, yet they are conditions which we wish to detect.

The agar 37-deg. count in connection with the gelatine often emphasizes these conditions, but many of them it would never detect alone. Furthermore, the agar 37-deg. count on the better class of waters is so low that it would take considerable audacity to say that there was a different interpretation to be placed on a fluctuation of 5 or 10 per cubic centimeter.

Gelatine shows a markedly wider range of fluctuation and consequently is a much better indicator of working conditions. On the other hand, the agar has no advantage as an indicator of sanitary conditions.

The writer wishes to add his protest against the replacement of gelatine 20-deg. C. counts with agar 37-deg. C. counts as standard procedure in water analysis. If it is possible to make only one count the gelatine count should be used, as it is more reliable and serviceable. Both counts, however, are desirable.

Purification of Water by Ultra-Violet Rays

By MAX VON RECKLINGHAUSEN
Of the R. U. V. Company, Inc., New York

FREEDOM from turbidity is one of the first requirements for a water which is to be sterilized by ultra-violet rays. When any suspended matter is interposed the bactericidal action cannot occur, for the microbe is in the shadow. In most cases it is necessary to filter the water before submitting it to the action of the lamps. As color in solution will absorb ultra-violet rays to a certain extent it is evidently better to free the water from coloring material before submitting it to the rays.

Sometimes water with little suspended matter may be more difficult to sterilize than water with far more suspended matter. Suspensions of purely mineral nature which do not enclose any microbes and to which few microbes are attached handicap the sterilization of the water very much less than suspended particles in similar water which are heavily covered with microbes and particularly so if microbes are enclosed in these particles, because it is then most likely that a very long and repeated exposure to the rays will be necessary to penetrate to the enclosed germ life. If the suspended matter is of smaller size than the germs, like colloidal clay, it acts more or less like color in solution, demanding simply some more illumination than clear water.

From the economical point of view the condition in which the water is submitted to the rays is evidently of the greatest importance for the ultra-violet ray sterilization system. Physically ideal water, or water without suspended matter, turbidity or color will need very little power in ultra-violet rays to become sterile.

Smaller installations are being equipped usually with paper pad filters. In large plants, naturally, the filter question is an engineering proposition, as is the question of choice between mechanical and slow sand filters. It seems that if the latter are chosen they can be speeded up to higher rates than is safe for biological filtration alone and still give a physically pure enough water for ultra-violet ray treatment. At the Luneville plant in France the water is filtered at three times the rate of biological filtration for that particular kind of water. In other plants the sand filters have been speeded up to 10,000,000 gal., 12,000,000 gal. per acre and we even tried with fair success 25,000,000 gal. per acre followed by ultra-violet ray treatment.

OPERATING COSTS

Operating costs will vary with the size and the running hours of the plant and the coefficient of safety for the ultra-violet ray treatment. According to the quality of the water it is expected in large plants which run 24 hours that the current consumption will vary between 30 and 125 kw-hr. per million gallons, allowing for a large safety coefficient. The labor charges are negligible, as the apparatus only needs an occasional cleaning and starting of lamp. Apart from this the lamps have to be repaired from time to time. When the water is of variable physical quality one will have to establish the plant so that all the lamps will be running during the period of least transparent water and only some of them during the period of greatest transparency.

If reliance is placed on chemicals to disinfect water, we must work close to and sometimes even over the limit of the amount which will not make itself objectionable by producing taste and odor in the water. There is an increasing opposition to the use of chemicals for the purification of water supplies. In the ultra-violet rays we have a system where we may choose our safety coefficient as high as we like. We may overdose our sterilization as much as we want without creating any objectionable features in the water, like taste and odor. The waterworks engineer has therefore in this system a much more satisfactory weapon against pollution than he ever had before.

Apportioning Fire-Protection Cost

By JOHN W. ALVORD
Consulting Engineer, Chicago

PROBABLY no one thing has contributed so much to the friction between private water companies and municipalities as the common and antiquated method of paying for fire protection at a fixed sum per hydrant. It is the purpose of this paper to point out some of the evils resulting from unscientific apportionment of fire-protection costs and show how they can be remedied.

The proper method of dividing the cost of public and private services is (1) to make a valuation of the plant, (2) to make a study of the value of a theoretical plant, sufficient in capacity for fire service only, and (3) a study of the value of a theoretical plant, capable of domestic service only. The sum of the (2) and (3) study will give a value much greater than the value of the combined plant, and the value of the combined plant should then be divided between fire service and domestic service in proportion to the theoretical values found necessary in each case.

In addition to this study it is necessary to inquire, in the same manner, into the operating expenses and divide the combined operating expenses between those chargeable to fire service and domestic service. Here it will be usually found that domestic service carries by far the higher charge.

Having found the total investment in each service, and computed its interest and depreciation, and added to this the proper proportion of operating expense, we have a proper and equitable division of annual revenue which should be derived as between the private consumer, on the one hand, and the public fire protection expense, on the other, and, as has been before said, it will be usually found that the public is not paying nearly enough for the public service and that the private consumer is bearing the burden. Here, therefore, is an opportunity for our public-service commissions to do some scientific rate-making, easing the burden particularly on the small consumer and placing it where it justly and properly belongs—on the general public.

But more than this can be done. When the proper amount for the public to pay for general fire protection is properly ascertained, it can be more scientifically applied than under the present irrational method, so that it will not result in the constant friction that the present method involves. The method of paying for fire protection at so much per hydrant is very common, but it is illogical and improper.

CAPACITY VERSUS NUMBER OF FIXTURES

Many of the bitterest contests between private companies and municipalities arise from this cause. All this source of contention and friction can be entirely avoided by a more proper method of payment for the public service, which will eliminate the objections above made, and place the burden of cost where it belongs—that is to say, in the capacity of the system for fire protection rather than on the number and spacing of outlet fixtures. Just as it is more equitable to sell water to the private consumer per unit of volume, by meter, rather than by number of outlets in the consumer's home, so is it more equitable to pay for fire protection per unit of capacity for fire protection purposes rather than mainly by unit of outlet fixture. In other words, let us pay the company for the amount of capacity it has in its pumping and distribution system, as well as for the number of outlets, and we will have destroyed most of the sources of friction that now exist.

The method that first suggested itself to the writer was to apportion fire-protection cost in part to pumping capacity, in part to pipe distribution capacity and in part to outlet, but on reflection it was seen that this triple division produced some complexity, which was not really needed, and which did not make for simplicity. It would be fully as effective to proportion fire-protection payments in large part on pipe distribution capacity and in small part on outlet fixture payments as to proceed in a more complex manner by including pumping capacity.

Having arrived at the proper amount to be paid the utility for fire protection it remains to apportion this amount in such a way that incentive is introduced to properly install mains and distribution system of the proper size, and hydrants of the proper spacing, so that the fire risks will be proportionately met in different sections of the town, and at the same time, so that extensions and additional hydrant rental will not be burdensome, and, in short, that all or most of the causes of friction between the municipality and the utility enumerated above will be removed. This can be done by paying for the fire service in large part per mile of pipe at rates depending on its capacity. For the purpose of having some regulating limit, it is also desirable to pay for the hydrants a small sum, which represents a reasonable return on their cost of installation, together with an allowance for their depreciation and annual maintenance. This in reality amounts to a nominal sum, generally not more than $5 to $8 per annum per hydrant.

The best unit which has occurred to the writer, that approximates in some degree the relative cost and capacity of mains sufficiently for this purpose, and is at the same time simple, is the inch-foot of diameter; that is, the number of inches of diameter 1 ft. long in a given system of distribution pipe. Thus, a 6-in. pipe is rated at 6 in.-ft. to every foot and a 12-in. pipe 12 in.-ft. to every foot.

Having found the total amount of money to be paid for fire service in a given system, we should deduct the amount that will be raised by the nominal hydrant rental before described, and we should also deduct the amount which will be raised from sprinkler systems and other especial methods of fire protection, and the remainder will be the amount to be paid by the city on the pipe capacity unit or inch-foot basis. The total number of inch-feet in a given system divided into the amount to be raised by this part of the protection will give us the cost per inch-foot. This will probably fall in most normal cases within 0.4 to 0.7 cent per inch-foot. The total annual payment, then, for any given pipe will be its diameter times the inch-foot unit times its length. Thus, if the inch-foot unit is half a cent, a 6-in. pipe will have a fire-protection rental of 3 cents per foot per annum and a 12-in. pipe will have a rental value similarly of 6 cents a foot annually.

To this amount computed for all the pipe lines should be added the nominal hydrant rental computed on the total number of hydrants, the total making the annual public fire-protection bill to the city.

Sewage-Disposal Works in European Cities

By GEORGE E. DATESMAN

Principal Assistant Engineer, Bureau of Surveys, Philadelphia

THROUGHOUT England, where rivers are comparatively small, sewage-disposal plants are undergoing reconstruction, and some form of bacterial treatment in beds is being added to the preliminary treatment. In some cases it has been found to be economical to turn the earlier works into scrap and build entirely new, as at Leeds. The disposal of sludge, except where it is carried to sea, has been generally inadequate. This is now considered by many the most vital problem. In Germany by several processes the problem of sludge disposal has been successfully solved.

As to the present status of the art the following statements may safely be made:

Sewage, even of exceptional concentration, can be effectively treated so as to secure a clear, odorless, sparkling, nonputrescible effluent before discharging into a stream the securing of which is a matter of cost and therefore an economic as well as a constructive problem.

Sludge resulting from sewage treatment can be rendered innocuous, practically inodorous, wholly unobjectionable, after which it may be dealt with in various ways.

Desired results may be secured by certain combinations of treatment at a fraction of the cost of other recognized scientific methods of treatment in satisfactory operations.

In general it is cheaper to treat water, when practical disinfection can be secured, by the admixture of from 0.3 to 0.7 p.p.m. of available chlorine instead of attempting to secure an uncertain comparative result in the treatment of sewage effluent by the admixture of from 2 to 6 parts, with a probability of having to resort to an equal amount of treatment for the water notwithstanding the sewage treatment.

Notable as being the best of their kinds are the following works, listed according to methods used: Hamburg, screening and dilution in the Elbe; Dresden, grit chamber and fine screening with dilution in the Elbe; Vienna, efficient collecting systems with dilution in the Danube; Frankfort-on-Main, grit chambers, screens, settling tanks, sludge dried in centrifugals, further dried by heat and burned under boilers to produce electric current; Wilmersdorf, primary settling tanks, percolating filters, secondary tanks, sand filtration, sludge in lagoons; Cologne and Dusseldorf, fine screening and tankage, with dilution in the Rhine; Berlin and Paris, farms; London, screens and tanks, with dilution in the Thames; Manchester and Sheffield, screens, tanks and contact beds; Birmingham, detritus tanks, settling tanks and percolating filters, sludge in lagoons; Salford, grit chambers, settling tanks, roughing filter, percolating filters, sludge mixed with chemicals, pressed and dried. Many others will outclass these when new works shall be in operation.

The quality of material and workmanship in sewer construction are superfine, due in a measure to mechanics' wages being one-third of ours. For the best developed screening appliances we must look to Germany. Tanks, both on account of economies in areas and scientific design for construction and operation, have been developed to a better standard in Germany than elsewhere.

Percolating filters and their operation may be best studied in England. Scientific experimentation on this system has been more thoroughly carried out in Paris. The investigations upon river dilution have been carried on more thoroughly in Germany. Sludge disposal, except where it is carried to sea, has not been solved in England. In Germany it has been satisfactorily solved by several methods.

The English sanitary world is hopeful that we are on the eve of developing a more intensive, economical and effective means of treatment than the percolating filter.

Washing Sand at Wilmington Filters

By E. M. HOOPES, Jr., Chief Engineer
and
JAMES M. CAIRD, Consulting Chemist

EXPERIENCE in washing sand with a Blaisdell machine at the slow sand filters in Wilmington, Del., has demonstrated that only 54 hp is required for the worst conditions of operation and 25 hp with short teeth and normal conditions. The power consumed in washing each bed of one-third of an acre in area has ranged from 80 to 175 kw-hr. and averages throughout the year approximately 120 kw-hr. The current consumed and power required are least when the shortest teeth are being used and reach a maximum with the longest teeth during the winter months, when the oil gear boxes tend to congeal and the operation of the machine is hampered by ice on the filter beds.

With a five-unit machine cleaning a strip 20 ft. wide one operator washes one-third of an acre in four hours, including transfer, and the bed is usually back in service within five hours from time of closing valves for washing process. This question of time element naturally affects the reserve area of plant that must otherwise be constructed in order to maintain normal rates of filtration throughout. The experience of the department has been that the time out of service by this method per acre area ranges from 7.5 to 10 per cent, while with the old method of hand scraping, ejecting and restoring, from 15 to 25 per cent of time is lost. It is apparent therefore that the construction cost, either for reserve area or plant capacity for low rates of filtration, is materially reduced by this improved mechanical method of filter cleaning. The method also had a material effect upon the cost of operating sand filters, effecting a reduction in our plant of approximately 50 per cent over the cost of operation of plants of the same capacity elsewhere, notably Philadelphia.

It was confidently expected when the plant was built that the operating cost would not be greater than $1 per 1,000,000 gal. exclusive of laboratory charges. We have not, however, been able to reach this low figure, the cost in the year 1910-11 being $1.08. As the operations of the plant require intelligent superintendence at all times, and as skilled mechanics must be employed continuously for operating the washing machine, notwithstanding that it is idle approximately 75 per cent of the available working time, it is evident that with a larger daily delivery in 1,000,000 gal., the above unit cost, which involves a considerable overhead or fixed expense, could be materially decreased.

Need of Filtered Water in New York

By GEORGE W. FULLER
Consulting Engineer, New York

It is rather curious that a ten-year-long attempt to give the citizens of New York a drinking-water supply of a quality such as is elsewhere demanded should have finally proved abortive. [After plans had been drawn and bids received for mechanical filters the project was killed by Mr. Mitchel on May 22, 1913.] Even more curious is it that the same officials who had voted to give New York a proper drinking water should later with the same information before them reverse themselves; that a Board of Aldermen, who but a short time before were considering a vote of censure on the executive department for their slowness in providing filtered water should a few months later repeal their ordinance and condemn the project; that the newspapers which had just stopped printing attacks on the city officers for their negligence in furnishing bad water should write to oppose the so-called steal when they were about to receive their demands.

The various phases of this matter have been clearly before the city officials for many years. It has never been represented as absolutely imperative. Neither is it imperative for the city to clean its streets daily, to maintain good smooth pavements, or to build a $15,000,000 office building. But it is certainly desirable that a city of the rank of New York should have a pride in doing these things; and it should have a pride in drinking a water for which it need not apologize as reasonably safe, at most times, and fairly decent-looking at most times.

Real safety we are not getting. Limited by the need of adding only enough hypochlorite not to spoil the taste, and with an organic content in the water varying from day to day, it is not physically possible closely to vary the hypochlorite dose so as to oxidize both the organic matter and the bacterial content and leave no objectionable excess. And conditions may arise at any time when the hypochlorite treatment will not be effectual in preventing the transmission of disease.

From an æsthetic viewpoint, Croton water is much of the time unobjectionable, but with turbidity running at times as high as 100 parts per million, color 60 parts per million, and sufficient microscopic organism to give unpleasant tastes and odors, our water supply is far from being a source of civic satisfaction.

Dual System of Distribution

By HENRY C. HODGKINS
Civil Engineer, Syracuse, N. Y.

There is a wide range in the cost of filtering water. Making due allowance for maintenance of plant, interest and depreciation, the cost would probably fall between 1 cent and 1.5 cents per 1000 gal. Assuming a cost of 1.2 cents per 1000 gal., the cost of filtering 1,000,000 gal. per day would be $12, and that of filtering 40,000 gal. per day, 48 cents, leaving $11.52 as the difference per day due to filtration. That is, if instead of filtering all of the water only the portion furnished through a dual system were filtered, the saving in the cost of filtration would amount to $11.52 per million. From this would have to be deducted the interest, depreciation and maintenance of the dual system, which in a medium-sized city would require not more than half of this saving, leaving the remainder for the additional cost of administration and saving to the department.

It is not claimed that the dual system should be universally adopted, but in places where there is a supply of naturally pure water sufficient to meet the culinary and drinking requirements it is urged that it be supplied through a separate system, leaving the larger requirements to be furnished from the nearest and cheapest source of reasonably clean water.

There are many instances where towns have outgrown their water supply. For just such an instance in the writer's practice it has been recommended and considered with favor that a duplicate system covering nearly half the town be installed, furnishing a water more suitable for manufacturing purposes but not fit for drinking. The original distribution is to supply all the needs of a portion of the residential section from the original source, and the smaller mains are to be extended, supplying all inhabitants with the excellent drinking water.

Because the industrial growth of the town has caused a demand beyond the capacity of the supply, but which is still sufficient for all house supplies for even a much larger town, should the best of nature's supplies be discarded and replaced by a filtered supply? The difficulties of supplying the ever-increasing demand with a potable water are such that all naturally pure waters should be conserved, and it would seem that it is time that the scientific planning of dual systems should be commenced.

The great argument against the dual supply has been the difficulty of controlling its use. It has been argued that poor people would use the cheaper water, and that many, especially new arrivals in the city, from ignorance would use the contaminated water. Every precaution should of course be taken to prevent the wrong use of either water. The drinking faucets should be of one handsome ornamental design, properly labeled. All other faucets should be of an entirely different and distinctive design. Greater care and inspection of all fixtures would be necessary, which would certainly be progress in the right direction.

Sewage Pollution of Boundary Waters

By ALLAN J. McLAUGHLIN, M.D.
Chief Sanitary Expert, International Joint Commission

In December, 1910, the writer was directed by the surgeon-general of the Public Health Service to investigate the sewage pollution of interstate and international waters, with special reference to the spread of typhoid fever. In 1911 he completed a sanitary survey of the entire watershed of the Great Lakes on the United States side of the boundary. Briefly the conditions found were as follows: An excessive prevalence of typhoid fever, especially in the winter and spring months, punctuated at intervals by explosive epidemics. This excessive prevalence of typhoid fever especially in the winter and spring months was due in greatest measure to the unrestricted discharge of sewage into interstate and international waters used as sources of public water supplies. Disaster followed the use of this sewage polluted water for one of two reasons, either a failure to purify or the inefficiency of the attempted purification.

The remedies suggested by the writer at that time were: 1. Safe water supplies, that is, water shown to be safe by daily bacteriologic examination. 2. Supervision and control of water supplies by the state to ensure efficiency and a safe effluent at all times. 3. Control of sewage discharge within permissible limits to prevent an unreasonable burden or responsibility upon filter plants. 4. Prevention of pollution from vessels.

In order to secure efficient and uniform results from the application of these remedies, he recommended that two sets of standards for water be formulated: 1. Standards for filtered or treated water. 2. Standards for raw water at the intakes.

The question of pollution of boundary waters was referred to the International Joint Commission by the governments of the United States and Canada. The reference was in two sections, the first dealing with the location, origin and extent of pollution and the second dealing with remedies. The Commission took up first the question of location, origin and extent of pollution (an abstract of its report was published in the Engineering Record of Jan. 31, page 126).

The writer's suggestion to the Commission in regard to proceeding with the second part of the reference consisted of three steps: 1. Secure the opinions of the most eminent sanitary engineers by submitting a list of questions. These opinions would form a sound basis for formulating a general policy and possibly some general minimum requirement. 2. Public hearings at which the municipalities could present their cases, including expert engineering testimony, plans and projects. 3. Employment of expert sanitary engineers for the formulation of special standards for each waterway in excess of general minimum requirements depending upon local conditions and necessities.

It is probable that the Commission will proceed somewhat according to this outline. In so doing and in carrying out the third step, formulation of special standards, we will realize the second of the essentials which are necessary for the protection of the public health, a standard for the raw water at the intakes. The standard for raw water at the intakes in each locality seems to be the only means by which an equitable adjustment can be secured between the two agencies in producing safe water, water purification and sewage treatment.

The greatest obstacle to proper operation and control of plants has been the difficulty of securing the right man to place in charge of the plant. The best type of man for this position is a graduate in sanitary engineering. He will not only be conversant with the mechanical details of the plant, but will be able to adjust his chemicals according to the constituents and needs of the raw water. Most important of all, he will be able to make daily bacteriological examinations to determine the efficiency of purification. Nearly all the disasters due to sewage polluted water supplies which have occurred were due to lack of daily bacteriological knowledge of the public supply or the inefficient operation of plants by unskilled men. The writer believes the employment of such a graduate is economy even in small cities.

Present-Day Water-Filtration Practice

By GEORGE A. JOHNSON
Consulting Engineer, New York

BETWEEN 1875, when the first American slow sand filter was built at Poughkeepsie, New York, and January, 1914, some thirty municipal filters of the slow sand type were put in operation or are now under construction; at this date such plants have a daily filtering capacity of about 840,000,000 gal., and are designed to serve a total population of some 5,500,000. Of this population 721,000, or about 12 per cent of the total, are located in New England, where Providence, R. I.; New Haven, Conn.; Lawrence and Springfield, Mass., are the largest cities having slow sand filter plants. Of the remaining population in the United States served by slow sand filter plants the bulk is centered in the cities of Albany, N. Y.; Philadelphia, Pittsburgh and Reading, Pa.; Washington, D. C.; Wilmington, Del.; Indianapolis, Ind., and Denver, Colo. The present population of these eight last named cities is about 4,000,000, representing about 73 per cent of the total. The remainder of the population served by this type of filters, equivalent to about 15 per cent of the total, is widely scattered through some twenty cities.

Specific comments, referring to the slow sand filter plants at Lawrence, Albany, Philadelphia, Washington and Pittsburgh have been set down to show the difficulties under which filters of this type are obliged to work when called upon to treat muddy waters. It is noted that in every instance, with the sole exception of Lawrence, the original design has been improved upon, and the preparatory treatment of the water made more complete. This was found imperative very shortly after the various plants went into service. Albany has added preliminary roughing filters; Philadelphia already had roughing filters which have proved but a limited success, and the use of coagulants is being considered in connection with preparatory treatment; Washington has added the coagulation process; and Pittsburgh has built roughing filters and is preparing to use coagulants and certain other chemicals.

Of other slow sand filter plants in the United States the Wilmington, Del., plant includes preliminary roughing filters, as does the plant at Steelton, Pa.; and Springfield, Mass., and Indianapolis, Ind., use coagulants. At practically all slow sand filter plants the final filtered product is sterilized with hypochlorites. At Lawrence the clarity of the Merrimac River water makes unnecessary the use of roughing filters, nor are coagulants used. The same is true at Providence, R. I., and New Haven, Conn., but with these exceptions it is becoming difficult to locate a slow sand filter plant which does not in some important respect depart from the original ideas of what constituted that system of water purification, or which does not in some way make use of certain inherent ideas upon which is based the rapid sand system of water filtration.

GROWTH OF RAPID SAND FILTRATION IN THE UNITED STATES

The first municipal rapid sand filtration plant was built at Somerville, N. J., in 1885, and between that date and January, 1914, upwards of 450 municipal filter plants of this type have been built or are now under construction in the United States. At this date the total daily capacity of filtration plants of the rapid sand type is 1,745,000,000 gal., and a total of nearly 12,000,000 people are being supplied with water so filtered. This population is distributed as follows:

GEOGRAPHIC DISTRIBUTION OF POPULATION SERVED BY RAPID SAND FILTER PLANTS

District	Per cent
New England States	2.0
Middle Atlantic States	22.0
South Atlantic States	10.0
Ohio Valley	26.0
Upper Mississippi Valley	13.0
Lower Mississippi Valley and Gulf	24.0
Rocky Mountain Region	0.2
Pacific Coast	3.2

In 1890 there were very few rapid sand filtration plants in operation in this country, and the decade which followed was fruitful with results of scientific investigation into the merits of the new process. The first series of public investigations were conducted at Providence in 1893, to be followed by even more elaborate studies at Louisville, Ky.; Pittsburgh, Pa.; Cincinnati, Ohio, and Washington, D. C. These investigations were in no small measure responsible for the wonderful growth of rapid sand filtration during the past 15 years, for the theory of the process was thoroughly worked out, and the whole idea becoming well understood was placed on a solid footing.

The type of construction changed abruptly about 1900, rectangular concrete tanks frequently replacing the circular wooden or steel tanks formerly used. The rectangular shape of the tanks made necessary the use of compressed air to agitate the sand layer while washing the filter, and later the application of wash water at high velocities. Both of these methods of filter cleaning had been used before, and therefore were not new in principle, but their use with rectangular shape filter tanks became more general, and supplanted the mechanical stirrers and wash water applied at low rate used in the older type of rapid sand filters, and from which they derived their original name, "mechanical" filters.

In the decade 1900-1910 many important rapid filter plants were built in this country, among the largest being Little Falls, New Jersey; New Orleans, Louisiana; Cincinnati, Ohio; Louisville, Kentucky, and Columbus, Ohio; the respective capacities of these plants being 32, 44, 112, 36 and 30 million gallons.

The average cost of building seven of the largest and most modern slow sand filter plants was $32,600 per million gallons daily capacity; and, likewise, the average cost of building six of the largest, and two medium size, rapid sand filtration plants was $12,100 per million gallons daily capacity. The average cost of operation and maintenance varied widely, of course, but averaged $2.86 and $4.04 per million gallons of water filtered by the slow sand and rapid sand systems, respectively. Adding these last figures to the fixed charge on the first cost of construction makes up the following totals:

Slow sand filtration	$7.33 per million gallons
Rapid sand filtration	5.70 per million gallons

SUMMARY AND CONCLUSIONS

To the writer the data at hand furnish indubitable evidence of the obsolescence of slow sand filtration and the steady rise of rapid sand filtration. When taking for comparison the relative growth of filtration in the United States by the two methods, it is clear that from the beginning the rapid sand method has predominated, until at the present day nearly 69 per cent of the total population of this country supplied with filtered water are served by rapid sand filters.

The first municipal slow sand filter in this country was built in 1875, and the first rapid sand filter in 1885. Since those dates the average annual number of people served by slow sand and rapid sand filter plants was 145,000 and 425,000, respectively, showing in this comparison the rapid sand filter practice to have been the superior of slow sand practice by some 200 per cent.

The decade 1900-1910 was a remarkable one in the line of water filtration development. Slow sand filters were built in many large cities, including Pittsburgh and Philadelphia, Pennsylvania, and Washington, District of Columbia; and rapid sand filter plants were placed in operation in such cities as Cincinnati, Columbus and Toledo, Ohio; Harrisburg, Pennsylvania, Louisville, Kentucky; New Orleans, Louisiana; and Paterson, New Jersey. All told, in that decade filter plants were built to serve a total of 8,945,000 people. When it is recalled that at the beginning of that decade only 1,860,000 people were supplied with filtered water, the contrast becomes particularly marked.

Since 1910 progress has continued. Filter plants have been built and placed in operation, or are now under construction in such cities as Baltimore, Maryland; Dallas, Texas; Evanston, Illinois; Grand Rapids, Michigan; Minneapolis, Minnesota; Portsmouth, Ohio; St. Louis, Missouri; Trenton, New Jersey; and elsewhere. All of these plants are of the rapid sand type, and it is significant that no new slow sand filter plants of size have been built since 1910, the work on filter plants of this type having been for the most part restricted to alterations and improvements whereby the efficiency of existing plants would be improved through the addition of coagulating or other preparatory processes.

From these facts it is readily seen that the rapid sand filter process not only predominates at this date, but bids fair to supplant before long, in a number of instances, the existing slow sand filter plants.

It has been demonstrated by actual experience that slow sand filters without the aid of costly preparatory treatment cannot efficiently purify such waters as those of the Hudson River at Albany, the Delaware at Philadelphia, the Potomac at Washington and the Allegheny at Pittsburgh. And even with preliminary filters there are cases where not only the appearance of the slow sand filtered water is at times unsatisfactory, but high bacterial efficiency at such times is obtained only with the aid of a supplementary sterilization process.

HYGIENIC EFFICIENCY

The hygienic efficiency of water filtration processes, measured by the reduction in typhoid fever, may be said to be about 70 per cent. There is little to choose between the average well built and operated slow sand and rapid sand filter, although such advantage as there is rests with the rapid sand filter process, which demands expert supervision, as should all means operated to the end of saving human life. Slow sand filters, even the simplest kind, and particularly those modified after the fashion of Philadelphia, Albany, Washington and Pittsburgh, also demand expert supervision. It is folly to consider the matter in any other light.

Questions of cost alone should never govern the manner of dealing with problems affecting public health and comfort. It so happens, however, that slow sand and rapid sand filtration cost about the same, with the advantage of lower cost usually in favor of the rapid sand process. A human life is customarily valued at $5,000, and a difference of even $15 per million gallons for water purification, one way or the other, amounts in a year's time to about the value of one human life.

Years of personal study and reasonably thorough familiarity with the performances of both types of water filters in scores of cities force the writer to the conclusion that in the efficient and economical solution of the vast majority of water purification problems in the United States the rapid sand filtration process is superior to the slow sand process. In view of the general trend during the past ten years or more toward the rapid sand process, and away from the slow sand process, it would seem that the writer is by no means alone in the conviction which he holds.

King James, it will be remembered, used to call for his old shoes, for they were easier for his feet, and there are some who still prefer the old type filter for the reason that they believe it will run itself, and is therefore less of a constant bother; but in view of the experience of the last twelve or more years the writer prefers the newer process of water purification because there is no question about it requiring constant skilled attention; because it is sure in its action; because it insures a minimum typhoid fever death rate coincidentally with the production of a clear, colorless and palatable water; and because it is no more costly than the older type of filtration process.

When one buys an expensive piece of machinery one doesn't turn it over to the questionable mercies of an unskilled mechanic. It is the same with a filter plant which costs far more money. The argument favoring the slow sand filter because it cannot go wrong is fallacious and nonsensical. If properly operated it also requires skilled supervision; and most of the so-called slow sand filter plants of to-day require a good deal of that.

Effect of Ozone on Algae Growths

By S. T. POWELL

Resident Chemist, Baltimore County Water & Electric Company

FOR the past five years the Baltimore County Water & Electric Company has been using ozone as a sterilizing agent to purify its Herring Run water supply. Herring Run is a small surface stream always carrying an appreciable amount of suspended matter. It has been necessary to treat the water many times a year with copper sulphate to keep down algæ and other vegetable growths. The water flows to the ozone plant direct from the reservoirs and is treated by means of aspiration, the falling water sucking the ozone directly from the generators, then passing through a mixing chamber to the suction well of the pump. This well is open to the atmosphere, so that the ozone not used up during passage through the mixing chamber is allowed to escape. As some of the algæ reduction seemed attributable to the agitation of the water in the mixing chamber and to the formation of scum in the suction well, laboratory tests were made to determine to what extent the algæ were killed by direct contact with the gas and the general effect resulting from ozonization of samples impregnated with such vegetable growths, especially with reference to color and odor.

Only low concentrations (0.20 to 0.30 gram per cubic centimeter) could be obtained; but we have noted that low concentration and long contact will usually insure as thorough oxidation of organic matter as the opposite combination. In the few tests where higher concentrations were used the Herring Run generators were called into play. The ozone was drawn through the water at the rate of 1 liter per minute, preventing violent agitation. The results of the experiments are shown in the table.

Where the ozone acted upon water containing living micro-organisms the characteristic odors were intensified, due to oxidation and disintegration of the plant, scattering the oil globules through the solution. As these were acted upon only after complete oxidation of the organic matter, waters containing micro-organisms were filtered before deodorization was attempted.

Some of the more important results follow: A faint fishy odor caused by algæ growths was removed in ten minutes with a concentration of 0.18 gram of the ozone gas.

A distinctive algæ odor arising from a sample of stagnant swamp water was greatly reduced in ten minutes with an ozone concentration of 0.20 gram per cubic meter and completely removed in thirty minutes. The same odor was entirely eliminated by a concentration of 1.69 grams in one minute.

AVERAGE PERCENTAGE REMOVAL OF MICRO-ORGANISMS BY OZONIZATION

	Per cent
Diatomaceæ	48.0
Cyanophyceæ	55.0
Chlorophyceæ	64.0
Fungi	46.2
Protozoa	41.5
Rotifera	32.8
Crustacea	30.0
Morphous matter	41.0

A solution of tannic acid was greatly increased in color and odor by two minutes' ozonization and a concentration of 0.20 gram of the gas. This condition is one of considerable importance, as it demonstrates one of the serious difficulties of color removal encountered in the operation of ozone sterilization.

The observations that have been made demonstrate clearly that certain forms of algæ are readily removed from water by direct oxidation, while certain other forms are entirely unaffected even with protracted periods of contact with the ozone. In addition to this it has been noted that there is an increase in the odor arising from direct ozonization of algæ that can only be removed after complete oxidation of the organic content of the water. Its deodorizing value depends on the oxidizability of the substance treated and the concentration of the gas, as well as the thoroughness of the mix.

Forty-Eight-Inch Force Main across Salt Marsh

By L. VAN GILDER

Superintendent of Water Department, Atlantic City, N. J.

TO afford a supply in case accident befall the wood-stave conduit now in use, a 48-in. cast-iron force main is being laid, at a cost of about $415,000, across 5 miles of salt marsh to the pumping station of the Atlantic City water department. The surface strata consist of soft muck from 5 to 25 ft. deep, constantly saturated with water to the grass roots and underlaid with sand, gravel and clay. The waters have a corrosive action on cast iron, while preserving wood. The main is therefore supported above the marsh on a foundation consisting of a double row of pine piles freshly cut from growing timber driven in pairs 3 ft. 6 in. center to center transversely and approximately 6 ft. longitudinally. Each pair of piles is capped with a reinforced-concrete bolster coved to fit the pipe, the bottom of the bolster being in breeching form to surround the piles and carried well below atmospheric action. By this arrangement every length of pipe is carried on two bolsters supported by four piles.

The bottom is so irregular that the piles vary in length from 20 to 45 ft., the weight of the hammer being sufficient to sink a large proportion of them from 20 to 25 ft. without a blow, while others require a water jet to set them deep enough for lateral stability.

To provide for expansion and contraction due to temperature changes each spigot is held not less than ¼ in. clear of the bell shoulder by inserting a gage ring in the joint. All gages are withdrawn after the pipes are placed in permanent position. All joints are poured with molten lead not less than 3 in. deep, and thus every pipe is carried on its own bolsters and is connected to adjoining pipes only by the lead joint rings.

As a safeguard against any possible chemical action between cast iron and concrete a 3/16-in. gasket of hard fiber is placed between pipe and bolster. Valves, manholes and blowoffs are placed at intervals of about 3000 ft. to facilitate internal inspection and repair.

At the city end the main crosses a navigable stream about 500 ft. wide, with a 200-ft. channel 12 ft. deep at mean low tide. This crossing will be made by a 48-in. bell-and-spigot inverted siphon with flexible joints at all deflection points, all laid on piling with timber caps. The entire structure is to be below the present stream bed.

The construction of this main contained so many uncertainties that it was deemed expedient to build it sectionally, the first contract for 6000 ft. being let in May, 1913. The work was so successful that the contract for the remainder, except the Thoroughfare siphon, was let Nov. 6, 1913. Every part of the work is being prosecuted with such vigor that we expect to have the main in service early in August.

The piling has not exceeded the estimate and the proximity of the Pennsylvania and Reading railroads has so facilitated the handling of materials that by the use of a work train and railroad derrick pipe, piling, gravel, sand and cement have been distributed at the approximate cost of handling on the city streets.

Testing of Check Valves

By J. WALTER ACKERMAN
Superintendent, Water Board, Auburn, N. Y.

IN the year 1908 the water department of the city of Auburn caused an examination to be made of the check valves of the mill and factory connections in the city of Auburn. It was found in many cases that even the locations of the valves were unknown to the factory, and some had been buried in the ground for a period of twenty years, without ever having been examined. The final result of this inspection was that each and every mill or factory using city water for fire protection, and having a polluted auxiliary supply, was required to install double check valves of a special design, and to be placed in a brick or concrete vault, accessible at all times to inspection and test. The regulations are that they shall be tested once every six months by the water department, and taken apart once each year and examined for defects or obstructions, and during this latter inspection a representative from the Associated Factory Mutual Fire Insurance Company is present to observe the condition of the valve and assist at the test. It is specified by the water department that the inspection and test shall be at the expense of the factory or mill where the tests are made.

The rules which control the installation of check valves and kindred appurtenances in Auburn are as follows:

"No water pipes in any building or premises supplied with water by the city shall be allowed to receive a supply of water from any other source except in case of pipes used exclusively for fire protection that conform to the requirements given below:

"No connection of the city mains shall be allowed with pipes having another source of water used for miscellaneous purposes except to a tank open to the air, nor shall such pipes have any physical connection with the pipes used for fire protection.

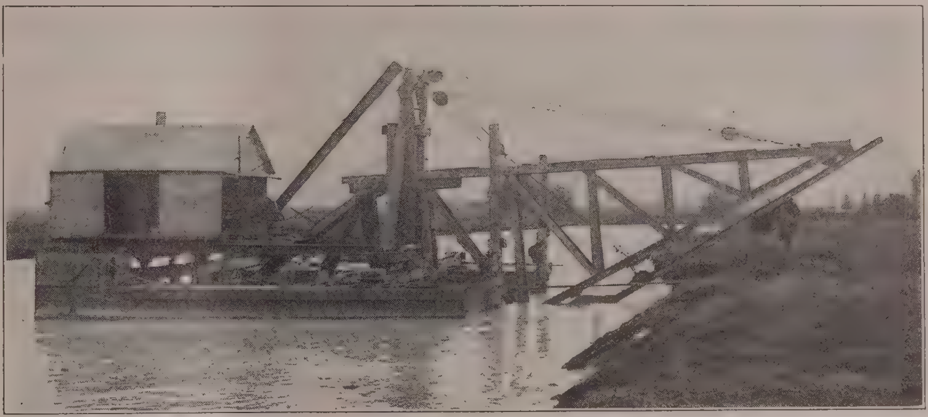
"Between the pipes used exclusively for fire protection and the city mains there shall be two check valves placed between two positive valves and located in an accessible brick or concrete pit, with such valves, gages and connections as may be prescribed by the water board for each installation and for the purpose of testing. The check valve shall be of a type approved by the water board. The check valves shall be kept in working order by the consumer and shall be at all times subject to inspection and test by the superintendent of water works, the cost of such test being borne by the consumer. The pure water pipes that have direct connection with the city supply, before the same have passed through the aforesaid double check valves, shall be painted blue; the pipes used exclusively for fire protection shall be painted red; and the pipes carrying the impure water and that used for miscellaneous mill purposes but not for drinking shall be painted yellow. These different systems of pipes shall not have any physical connection with each other in the mill or factory.

"There shall be placed on the discharge main of the fire pump a check valve and positive gate. The latter shall be kept closed and sealed by the superintendent of water works, and opened only in case of fire or for the purpose of occasionally testing the fire system, but notice of such occasional test shall be given to the superintendent of water works, and during the test the positive gate between the fire system and the city main shall be kept closed. During the weekly trials of the pump prescribed by the fire underwriters the water shall be discharged either through the relief valves of the pump or through hose connections from the pump."

Boat for Sloping Canal Banks

Novel Outfit Specially-Devised to Follow Suction Dredges and Shape the Slopes on New York State Barge Canal Work at Rome

A SPECIAL sloping boat built to follow the dredges on the New York State Barge Canal work at Rome has proved successful in every way, the only criticism made by the contractors who devised and built it being that it works so rapidly that it cannot be kept busy continuously. The machine is shown in the illustration. It consists of two scows joined above the water surface and separated by two trusses, so as to leave a well between them. These trusses carry a track on which a carriage operates, having a lateral motion of 30 ft. for each setting of the boat. From this carriage is suspended a double cantilever truss carrying on its lower chord a double track which conforms to the contour of the slope desired from the surface of the ground to the bed of the canal, and extends outwardly from the scows and over the bank.



Sloping Boat Devised for New York State Barge Canal Work at Rome

A cutting bucket of 1-yd. capacity, in many respects similar to a dragline bucket, travels on this track, the cutting edge being below and against the bank to be trimmed. The back of the bucket has two flap valves opening inward for the purpose of permitting water to enter and assist in washing out the material from the bucket on its return or upward travel. At a point under the scows and beyond the toe of the slope, the tracks on which the bucket runs are so deflected as to result in the bucket being tilted and dropping out its load, the dredge having previously excavated below the established elevation of the bottom of the canal a depth sufficient to receive this material and leave nothing above the established elevation.

ADJUSTABLE TO ANY SLOPE

As there are several different slopes specified on different parts of the Barge Canal work, the arm is made adjustable to any slope. As originally designed, the canal bank at this point was to have a 1 on 2 slope under water, a strip of pavement, or wash wall at the water line, a level berm above this, and a 1 on 1½ slope above that. It was proposed to build an irregular track to fit, so that the machine would cut down the steeper slope, level the berm, excavate for the pavement, step outward again, and finish the lighter slope under water. This was considered entirely practicable, but the Barge Canal design was changed to a single uniform slope of 1 on 4 and the track for the scoop was simplified accordingly.

This machine was designed by A. S. Robinson, engineer of Grant Smith & Company & Locher, of New York City, who built and operated it for the extensive work at Rome described in the issues of Feb. 21, March 7 and April 18. The M. A. Talbott Company, of Baltimore, was the general contractor.

Fundamentals of Drainage

Abstract of Remarks by Nathan C. Grover, Chief Hydraulic Engineer, U. S. Geological Survey, before the Drainage Congress at Savannah

IN starting the new form of national activity which is now proposed—the drainage of swamp lands—a serious attempt should be made to put the work on a sound basis and to avoid as far as possible the mistakes made in irrigation. Irrigation and drainage have much in common and are liable to practically the same dangers—in fact, in many localities not only in the West but in the East drainage must be accompanied by irrigation. General methods for financing and managing drainage projects having been devised, the data necessary for designing the drainage system may usually be collected in such a field survey as can be made within a period of a few months, except as regards the quantity of water for which the canals must be designed.

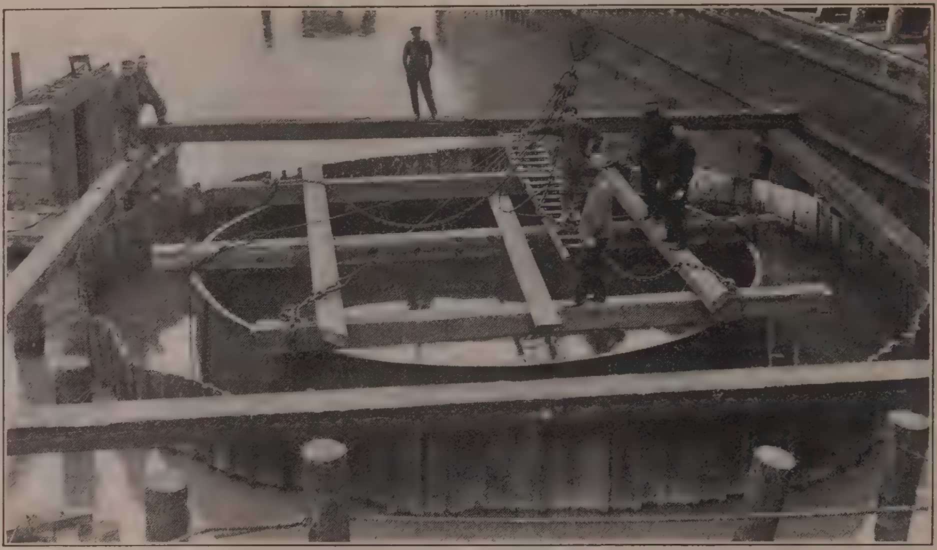
This last determination, however, involves the collection of records extending over long periods. No such determinations have been made for large swamp areas in the southeastern States. Designs for drainage construction have had no other basis than general records of precipitation and evaporation, and the result has been that drainage systems have been designed and partly constructed with insufficient capacity to accomplish the desired results. Disappointment may follow for thousands of small purchasers of land who have been assured that the land would be speedily drained.

IMPORTANCE OF FUNDAMENTALS

The failure of one project will have its baneful effect on all, and the failure of a considerable number will make the financing and colonization of others impossible. The result will be a situation like that which is to-day preventing the financing and colonization of irrigation projects, in which the intrinsic soundness and value of individual projects will be completely lost sight of in the general distrust caused by the failure to recognize the fundamental factors controlling all such developments.

Several years should be devoted to the determination of this fundamental factor before there will be assurance that the records cover the ordinary range of conditions caused by fluctuations in precipitation.

The measurement of runoff need not and should not wait until arrangements have been made for financing and managing drainage works, but, because of the long time required, should be begun at once. In other words, such preliminary data should be collected that the actual work of development may immediately follow the passage of a national drainage act, in the same way that the prior collection of stream-flow data by the Geological Survey made it possible to begin the construction of irrigation projects immediately after the passage of the national reclamation act.



Steel Form for Center Pier Set in Position, with Cradle Attached

Building New Pennsylvania Bridge over the Maumee River at Toledo in Record Time

Part II—Steel Was First Employed for the Interchangeable Submerged Forms, but Afterward Rejected in Favor of Wood

By R. C. MILLER

Division Engineer, Pennsylvania Lines West of Pittsburgh Toledo, Ohio

ASIDE from the two abutments, the design of the new Maumee River bridge of the Pennsylvania Lines West of Pittsburgh, the foundation work of which was described in the issue of May 2, page 492, was such that only three separate forms would be needed for all concrete work from tops of footing courses to undercopings. It was accordingly decided to use steel forms. These did not prove satisfactory, however, and were abandoned after three of the eight piers had been built. The wooden forms which replaced them were very successful.

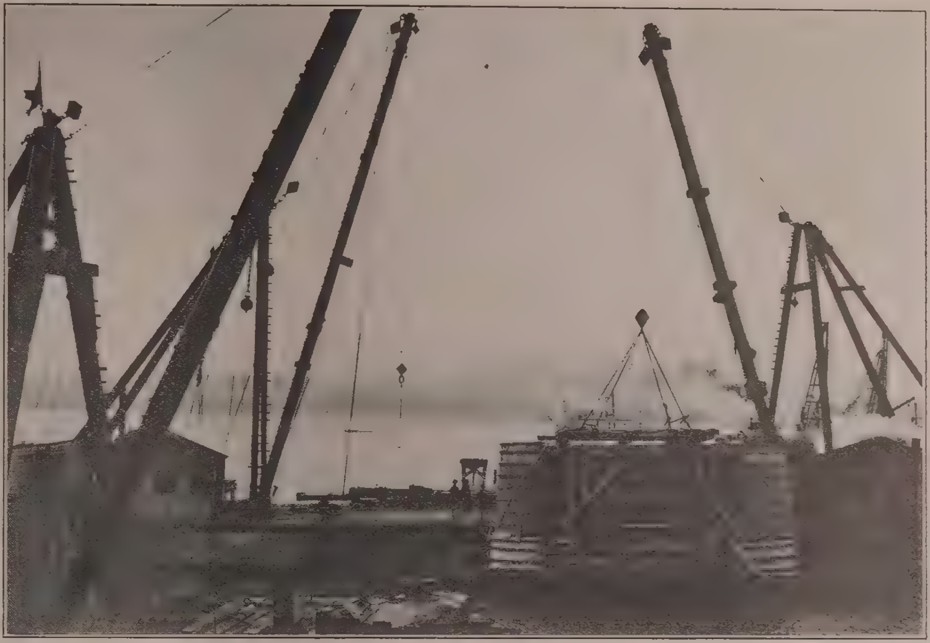
Pier 5, the drawspan pier, was circular in shape, and one form 37 ft. high with a diameter of 29 ft. at the bottom and 23 ft. at the top would have to be built for this pier alone. Piers 4 and 6, at each end of the draw, were alike, consisting of two parallel sides for the center section, with Gothic ends. One form 53 ft. long, 21 ft. wide and 36 ft. high could be used for both of these. Piers 0, 1, 2, 3 and 7 were alike, with two parallel sides for the center section, the downstream ends being half circles and the upstream ends of Gothic shape. One form 42 ft. long, 14 ft. wide and 36 ft. high could be used for them. On all, with the exception of pier 5, there was a steel icebreaker to be embedded in the concrete on the upstream end.

Because of this symmetry in design and on account of depositing concrete under water in open caisson without pumping it was decided to use the three steel forms. These forms were built in nine horizontal sections 4 ft. high, six of the sections being assembled on timbers laid across the cofferdam, lifted by derrick scows while the timbers were removed, and then lowered into place. Due to carelessness in the shops, the sections did not fit well and were difficult to hold in line, and the general workmanship was such that considerable field work was necessary before they were ready for concrete. Also the design of the forms was such that each section was bolted together with angles facing each other and riveted to the respective sections, thus necessitating considerable work before the forms could be released from one pier to be used on another.

STEEL FORMS ABANDONED

Erection of the form in the open above the cofferdam was not difficult. When lowered to place in form 20 to 24 ft. of water, and filled with concrete, however—thus bringing horizontal pressure against the form and a constant strain upon the bolts—the work was not so easy. Add to this the fact that this work must be done by divers, in water that is never clear, relying entirely upon the sense of touch, and the question of the removal of these forms becomes paramount, both from the expense incurred and the time involved. It quickly developed that repeated use of these steel forms would defeat completing the work on schedule time, and, therefore, after they were used on piers 2, 4 and 5 they were abandoned, wooden forms being used for the remainder of the work.

The wooden forms were built in three horizontal sections 12 ft. high. The curved ends were built of 1-in. vertical tapered sheeting held together with 2 x 4-in. oak bands bent to the exact line of the curve. They were joined to the straight part of the form by 2 x 3/16-in. iron straps passing around the curved ends and over the oak bands. On the ends of the straps were welded threaded bolts by which the curved and straight sections were drawn together with turnbuckles, also allowing the separation of these parts when removing the forms from the completed work. On the straight part of the forms the 1 x 12-in. sheeting was placed horizontally and held together and stiffened by 2 x 6-in. vertical studding spaced 18 in. on centers. These in turn were held in place by 3 x 5-in. waling on each side spaced 36 in. vertically, through which pipe was run. This pipe took compression while the forms were be-



Lowering Wood Form to Place at One of the Piers

ing handled, preventing collapse and tension when concrete was being run. The 12-ft. sections were joined by the lapping and bolting together of the alternate 2 x 6-in. studding, one bolt being used for each splice connection. To release the section a diver had to loosen the bolts. While the two top sections were in each case reclaimed and used on other piers, new bottom sections were made for each pier to obviate the difficulty in releasing the old ones without breaking them.

For the east abutment the wooden form was very large, consisting of the center part, 25 ft. long, 20 ft. wide on the bottom, 10 ft. wide at the undercoping and 38 ft. high, and two wings, each 35 ft. long, 12 ft. wide and 27 ft. high at the end. This form was built first in two 12-ft. horizontal sections, constructed on land and handled to place by derrick scows. Inasmuch as the extreme length of one of these sections was practically 95 ft., each section was further subdivided into three sections, consisting of the main body and the two wings. The details of construction were the same as those used for the wooden pier forms. This brought the concrete up above water, the remaining 14 ft. of height being obtained by building the form in place after running the tremie concrete. The form for the west abutment was similar, but much smaller. One horizontal section 18 ft. high, constructed as a unit on land and handled to place by derrick scow, brought the concrete above water. The remainder was handled the same as for the east abutment.

COPINGS

When the concrete reached the undercoping the wood form was removed and a narrow platform built entirely around the pier at the correct elevation. Upon this a 2-ft. coping form, built on land, was set and lined to place, the platform constituting the bottom of the form to take care of the 6-in. overhang. Owing to the excessive depth of the grillages on top of the masonry it was necessary to arrange for holes in the coping concrete to allow the future placing of 2-in. anchor bolts without drilling. Consequently, after lining to place and securely fastening the form, a large template constructed on land and carrying pieces of the required length of 6-in. stovepipe accurately located was set on top of the coping form and lined to place. The concrete was then poured. A total of 136 anchor-bolt holes were thus provided for in the concrete.

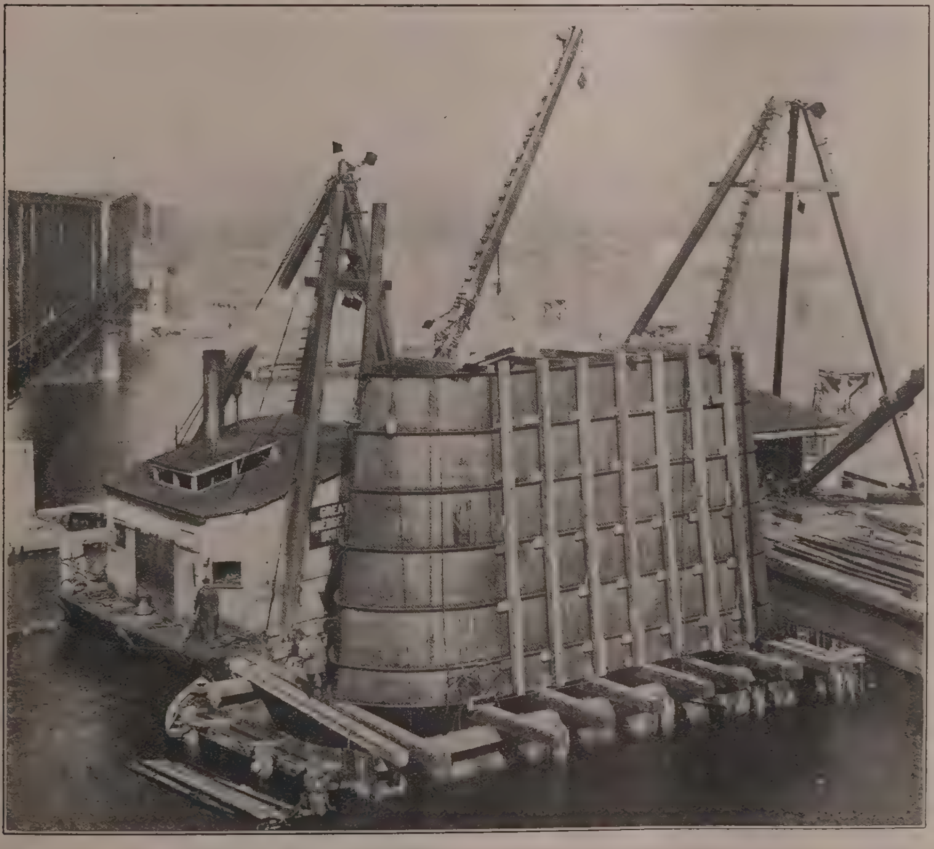
The coping form for pier 5 was circular in shape, 4 ft. high and of such peculiar construction that it was necessary to build this form entirely in place after pouring the concrete to the undercoping. Since the entire bridge is on a 0.6 per cent grade, the drawspan was necessarily on this grade. At the same time it was required to swing the draw either 180 or 360 deg. and handle traffic in either position. Hence the concrete on the pivot pier had to be surfaced in a plane parallel to the gradient line of the bridge. The cutoff of the top of the circular coping form was determined by setting a level over the center of the pier and bringing the horizontal axis of the bubble tube into a plane parallel to the 0.6 per cent gradient. This method furnished an absolute check on elevations established and at the same time gave an accurate location of the cutoff on all points of the circle.

Handling and setting of the forms to line and grade was in itself a delicate piece of work. In determining the height of the vertical sections of the wood forms, consideration was given to the fact that the depth of water over the footing course varied from 16 ft. to 22 ft. Consequently, two 12-ft. sections for the east abutment and all piers would allow the form when set to stand about 2 ft. out of water, this amount representing the normal fluctuation of the water level.

HANDLING AND SETTING FORMS

With the form completed a derrick scow lifted the bottom section and set it on the scow. To do this, a cradle as wide as the bottom section, built of heavy timbers, was placed on top of the form and lashed to place at four corners, applying the strain of the load when lifted to various points on the side of the form by lashing to the 3 x 5-in. wale strips. This cradle arrangement was used most successfully with all forms handled. The cradle was lifted by the boom and balanced by means of cables from the four corners. After setting the first section on the scow, the second was picked up and set to place on top of the first.

While the scow was being transferred to the cofferdam the two sections were fastened together and the cradle was arranged to handle both sections at once. These were lowered into the water, the



Derricks Getting Ready to Lower Six Sections of a Steel Form cradle was removed and buoyancy was overcome by setting steel rails on the form vertically between the waling and the sheeting, enough load being applied to make the form settle to a firm bearing on the footing course. Next 12 x 12-in. timbers were laid across the cofferdam, on the falsework used in driving the foundation piling, at each end of the center section of the form. From these timbers the form was suspended by steamboat ratchets and gradually worked to correct line and elevation. With 22 ft. of the form under water and hanging from the top alone, it was necessary to use extreme care in this final adjustment in order that the pier should have the correct batter when completed. When set the top was braced from all sides against the cofferdam and the bottom held to place by packing sacks filled with a strong mixture of cement and sand around the entire inside edge of the form. These sacks also took up any irregularities in the footing course and prevented loss of concrete. After the concrete was run to within 2 ft. of mean lake level the top 12-ft. section of the form was set in place.

CONCLUSION

While steel forms have been successfully used to a considerable extent in repetition of design, in all such cases the concrete has been run in the dry and the setting and dismantling of the forms were not difficult. From the experience obtained in handling both steel and wood forms in this work the superiority of the latter for work handled in this manner was clearly shown. They were not only more easily handled and adjusted to line and grade but were more quickly removed from completed work and with but little damage.

[In an early issue Mr. Miller will describe the placing of the concrete by the tremie method, and the difficulties encountered with laitance, both in minimizing its formation and in removing what could not be prevented from forming. His experience with the Toledo bridge has suggested remedies for future construction which he will set forth.—Editor.]

Ortho-tolidin Test for Free Chlorine

IN 1909 Dr. L. P. Kinnicutt presented as testimony in the suit of Jersey City vs. Jersey City Water Supply Company results of tests on the use of ortho-tolidin as a reagent for oxidizing bleach and other solutions. As the testimony is not readily available and several chemists have recently nearly "rediscovered" ortho-tolidin, W. F. Monfort prepared an abstract of the 1909 testimony for the laboratory section of the American Water Works Association.

A yellow substance is formed by nascent oxygen, ozone, sodium nitrite, nitrous acid, magnesium persulphate, ferric salts, lead and sodium peroxide and several others. Minimum quantities, detectable in parts per million, are: Sodium nitrite, 2; ferric chloride or ferric alum, 20; potassium permanganate, 0.2; potassium chromate, 2; bleach or chlorine solutions, 0.068. The ortho-tolidin was prepared with sulphuric acid from commercial chemical. When made up with hydrochloric acid and chemically pure ortho-tolidin, four times the above accuracy is possible, making the reagent as delicate as the starch-iodide method.

Dr. E. Bartow remarked that small quantities were best tested by ortho-tolidin and larger quantities by starch.

Vents on Steel Pipe Lines

Determination of Capacity of Air Inlets at Summits to Prevent Formation of Vacuum Sufficient to Cause Collapse of Conduit

By M. L. ENGER and F. B. SEELY

Of the Department of Theoretical and Applied Mechanics, University of Illinois

FAILURE of steel pipe lines has shown that such pipes are not effective in resisting external pressures. In order to prevent the formation of a vacuum sufficient to cause the collapse of any part of the pipe line, air-inlet valves are used at the summits or at other points at which the pressure is likely to become less than that of the atmosphere. These valves also allow the air to escape while the pipe line is being filled with water. As far as the writers are aware, no rational method to aid in the determination of the required size of valve opening has been published. The object of this paper is to present a solution of this problem.

A vacuum may be formed in the pipe in one of the following ways: (1) By the sudden opening of a blowoff valve or the valve at the end of the pipe line; during the time that the water in the pipe is accelerating the pressure in the pipe at the summits may become less than atmospheric; (2) by too large a blowoff valve, causing one or more of the summits of the pipe line to be above the hydraulic gradient when the valve is fully opened; (3) by closing the intake valve, or any other valve, when a valve at a lower elevation is open, or (4) by the sudden rupture of the pipe line at a point of low elevation, causing a great rush of water away from the adjacent summits, which will draw the hydraulic gradient below the pipe at these points.

THIN PIPE UNDER COMPRESSION

When a perfectly round, thin pipe is subjected to an external normal pressure, a compressive stress will be developed at all points in the circumference. The ring of metal composing a part of the pipe therefore acts as a column carrying an axial load. When the external pressure reaches a critical value the pipe will fail by collapsing, just as a long or slender column will fail by buckling when the load becomes great enough. Euler's formula for columns with free ends is

$$W = \pi^2 EI/l^2$$

where W is the axial load, in pounds, which will cause the failure of a long column; E, modulus of elasticity of the material of which the column is composed, in pounds per square inch; I, least moment of inertia of cross-section of column, in inches^4, and l length of column, in inches. Euler's formula does not apply unless the length of the column is great as compared with its least radius of gyration.

Consider a ring included between two planes perpendicular to the axis of the pipe and 1 in. apart. The total stress on every cross-section of the ring due to the difference of the external and internal pressures is $(p_2 - p_1)d/2$, where p_2 and p_1 are respectively the external and internal unit pressures, in pounds per square inch, and d is the diameter of the pipe in inches. This stress corresponds to the axial load W on the long column of Euler's formula.

The least moment of inertia of the cross-section of the ring is $t^3/12$. When the pipe collapses there will be four points of counterflexure, or points of zero bending moment, as is seen from an inspection of Fig. 1. The length of the free-ended column, equivalent to the part of the ring between the points of zero bending moment, is, therefore $\pi d/4$. Substituting in Euler's formula and solving,

$$p_2 - p_1 = (8\,E/3)\,(t/d)^3 \qquad (1)$$

When $E = 29{,}000{,}000$ lb. per square inch.

$$p_2 - p_1 = 77{,}300{,}000\ (t/d)^3 \qquad (2)$$

This formula gives the maximum difference between the external and the internal pressures which the pipe can withstand. If the pipe is not exactly round or if the material is not homogeneous, less pressure will be required to cause the collapse than is given by the above formula.

EXPERIMENTAL FORMULA

No experiments have been made on large pipes by which the accuracy of the above formula can be tested. Experiments, have, however, been made to determine the external pressure required to collapse boiler tubes and well casings. The most elaborate are those of Prof. R. T. Stewart, "Collapsing Pressures of Bessemer Lap-Welded Tubes, Three to Ten Inches in Diameter," "Transactions," American Society of Mechanical Engineers, 1906, Vol. 27, page 730. Another series was made in the same year by Prof. A. P. Carman and M. L. Carr, "Resistance of Tubes to Collapse," Bulletin 5 of the University of Illinois Experiment Station. The experiments of Carman and Carr supplement and corroborate those of Professor Stewart.

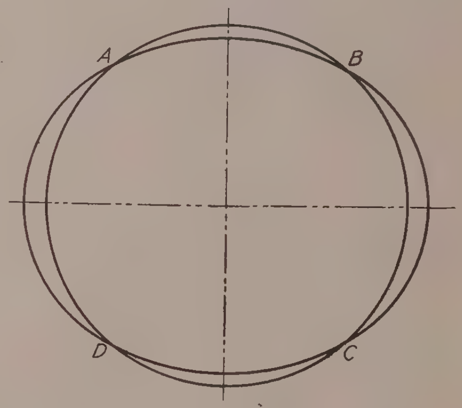


Fig. 1

Since the experiments were made to determine the strength of boiler tubes and well casing, the tubes upon which the experiments were made were relatively thick. The results of the experiments fall into two groups, according as t/d is greater or less than 0.025. Large steel pipes will almost always fall into the last group. Stewart and Carman give approximately the same formula for the collapsing pressure on pipes of this group:

$$p_2 - p_1 = 50{,}200{,}000\ (t/d)^3 \qquad (3)$$

It will be noticed that the coefficient determined by experiment in equation (3) is about 35 per cent less than the value determined from theoretical considerations alone —equation (2).

The experiments were made on cold-drawn and lap-welded tubes less than 10 in. in diameter. Because of the rational basis of equation (3) it may reasonably be expected to apply to similar pipes of larger diameter having smaller values of t/d than those experimented upon. In the case of riveted pipe with lap joints, the effect of the longitudinal joint will be to make the pipe collapse at a lower value of $p_2 - p_1$ than given by equation (3). The effect of the circular or transverse joints is to stiffen the pipe against collapse, but the increase in strength due to this cause is not as great as might at first be thought, probably being less than 4 per cent. A riveted pipe in most cases would not be as truly circular as the tubes upon which the experiments were made. As the size of the pipe increases the effect of the weight of the shell tending to cause the collapse of the pipe increases. All factors considered, a riveted pipe with lap joints is probably not as strong in resisting external pressure as the small lap-welded pipe having the same t/d, and it may be considerably weaker, especially if the diameter is large. Riveted pipe with butt joints is, no doubt, stronger than similar pipe with lap joints, but the difference is not important.

Assuming that riveted pipes are as strong as indicated by equation (3), the least diameters which pipes of a given thickness can have and fail by collapsing when subjected to various values of $p_2 - p_1$ have been calculated, and the results are given in Table 1.

TABLE 1—PIPES OF VARIOUS THICKNESSES AND DIAMETERS WHICH WILL FAIL UNDER THE GIVEN PRESSURE DIFFERENCE

p_2-p_1 lb. per sq. in.	$\frac{t}{d}$	Least diameter of pipe which will collapse		
		$t=\frac{1}{4}$ in.	$t=5/16$ in.	$t=\frac{3}{8}$ in.
14.7	0.0066	38.0	47.5	57.0
10.0	0.0059	47.5	53.0	64.0
9.0	0.0056	44.5	56.0	67.5
8.0	0.0054	46.5	58.0	70.0
7.0	0.0051	48.5	61.3	73.0
6.0	0.0049	51.5	64.0	77.0
5.0	0.0046	54.5	68.0	82.0
4.0	0.0043	59.0	73.0	88.0
3.0	0.0039	64.5	80.0	97.0
2.0	0.0034	74.0	92.0	111.0
1.0	0.0027	93.0	116.0	139.0
0.5	0.0021	117.0	149.0	176.0

An examination of the table will show that even when the pipes are assumed to be as strong as lap-welded pipe a relatively low vacuum may cause the collapse of the pipe. It is therefore evident that careful attention must be given to the choice of air-inlet valves for a large pipe line.

APPLICATION OF AIR VALVES

The amount of air which must be admitted to keep the pressure from going below a given value must be such as will, at the given pressure, replace the water which has run away from the section. The air-inlet valves at the summits of the pipe line should be designed to prevent the pressure from becoming less than the critical value for the severest conditions. Let Fig. 2 represent a pipe line. If the rupture should occur at D there would be a rush of water from C to D and from E to D. The water between the reservoir and C would accelerate slowly. A vacuum would therefore be formed at C. In order to prevent the pressure at C from going below a predetermined value, a volume of air equal to the difference of the volumes flowing from C to D and from A to C must be supplied. The column of water between A and C is acted upon by the head h_1, which is the sum of the difference in elevation of A and C and the allowable vacuum at C. The column of water between C and D is acted upon by a head h_2, which is the height of C above D less the vacuum at C. The head h_1 will be nearly constant, but the head h_2 will decrease as the level of the water in the section CD falls. In most cases the lowering of the water in CD can be neglected, but in any case a reasonable assumption can be made as to the value of h_2. The greatest amount of air required will be when the flow in the two branches has reached its maximum value, and may hence be computed from the Chezy formula, $v = c\sqrt{rs}$, using $c = 110$ for riveted pipes.

$$Q = Au = A\,(v_3 - v_1) = 55A\sqrt{d} \times (\sqrt{s_2} - \sqrt{s_1}) = 55\,A\sqrt{d} \times (\sqrt{h_2/l_2} - \sqrt{h_1/l_1}) \qquad (4)$$

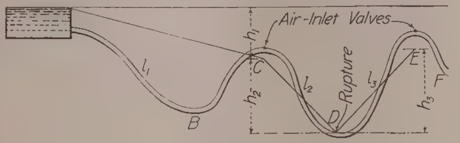


Fig. 2

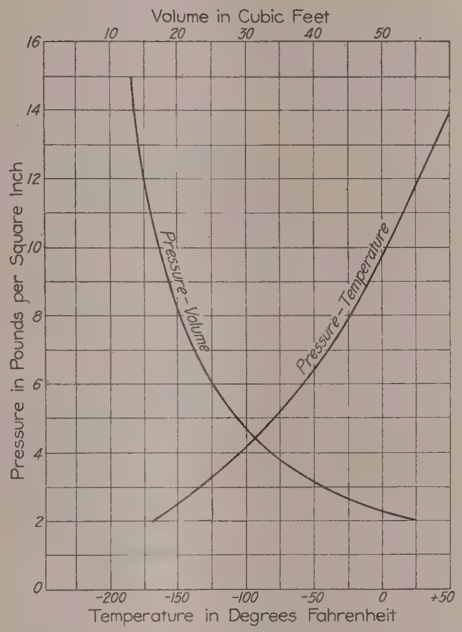


Fig. 3—Adiabatic Expansion of Air

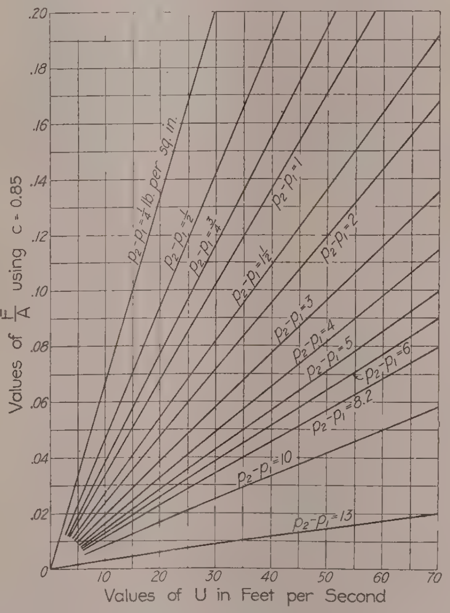


Fig. 4—Diagram Using c = 0.85

The amount of air which must be supplied under other conditions can easily be determined in a similar manner. It will be easily seen from an inspection of Fig. 2 that a rupture at D would cause a severe vacuum at E, because there would be no flow from F to E to help supply the volume, and if there was a flow from E to F at the time the rupture occurred it would be necessary to supply air to make up for this volume as well as for the volume flowing from E to D.

SMALL PRESSURE DIFFERENCES

The area of valve opening required for the entrance of the volume of air, Au, into the pipe may be determined from the equations for the flow of air through orifices, provided the coefficient of discharge for the valve opening is known. If the difference of pressure causing the flow of air is small (not more than 3 lb. per square inch) the flow may be treated as if the air were incompressible.

The theoretical velocity v of flow through the valve opening, for small pressure differences, is $\sqrt{2gh}$, h being in feet of air. Then

$$Q = cF\sqrt{2g\,(P_2 - P_1)/w}$$

where Q = quantity of air discharged through valve, in cubic feet per second; c, coefficient of discharge; F, area of valve opening, in square feet; g, acceleration due to gravity, 32.2; P_2, the absolute pressure of the air surrounding the pipe, in pounds per square foot; P_1, absolute pressure within the pipe, in pounds per square foot, and w, weight of 1 cu. ft. of air at pressure P_2 and temperature T_2 (0.0764 lb. at 60 deg. Fahr. and atmospheric pressure).

Since the quantity of air flowing through the valve must be equal to Au,

$$Q = Au = cF\sqrt{2g\,(P_2 - P_1)/w}$$

and

$$F/A \doteq u/c\sqrt{2g\,(P_2 - P_1)/w} \qquad (5)$$

From this equation the ratio of the area of the valve opening to the area of the cross-section of the pipe may be determined for any value of u, when the pressure difference is small.

LARGE PRESSURE DIFFERENCES

When the flow of air is due to pressure differences which are not small as compared with the absolute pressure of the air, the expansion of the air should be considered in calculating the flow.

The time required for air at pressure P_2 to expand through the valve is very short. The expansion may therefore be considered adiabatic. The temperature change may be very great, as is shown by the pressure-temperature curve in Fig. 3. As soon as the air enters the pipe its temperature gradually rises toward that of the pipe and water. Air, however, is a poor conductor of heat. The time required for this reheating is very long compared with the time of expanding through the valve, and since the collapse of the pipe would be rapid if the pressure difference became sufficient to start the failure, the rise of temperature of the air after entrance in the pipe, with the corresponding increase in pressure, should be neglected. The effect of neglecting the rise in temperature is to give results on the safe side.

When P_1/P_2 is greater than 0.528, and assuming adiabatic expansion,

$$W = 15.01\,cF \times \sqrt{(P_2/V_2)\,[(P_1/P_2)^{1.43} - (P_1/P_2)^{1.71}]} \qquad (6)$$

where W is the weight of air discharged through the valve per second. Therefore

$$Q = WV_1 = 195\,cF\,(P_2/P_1)^{0.715} \times \sqrt{(P_2/V_1)\,[(P_1/P_2)^{1.43} - (P_1/P_2)^{1.71}]} \qquad (7)$$

and

$$V_2 = V_1\,(P_1/P_2)^{(k-1)/k}$$

where $k = 1.4$ for air, V_1 and V_2 are the volumes of air, in cubic feet, corresponding to the pressures P_1 and P_2, and all other symbols have the same meaning as in equation (4).

Since Q must equal Au,

$$F/A = u/195\,c\,(P_2/P_1)^{0.715} \times \sqrt{(P_2/V_2)\left[(P_1/P_2)^{1.43} - (P_1/P_2)^{1.71}\right]} \quad (8)$$

When $P_1/P_2 \leqq 0.528$,

$$Q = 50\,cF\,(P_2\ P_1)^{0.715}\sqrt{P_2/V_2} = Au \quad (9)$$

and

$$F/A = u/50\,c\,(P_2/P_1)^{0.715}\sqrt{P_2/P_1} \quad (10)$$

The value of c to be used in the above equations probably varies for each type of valve, and should be determined. Experiments on the flow of air through orifices, short tubes and annular valve openings are not numerous. From a few experiments, however, made under rather widely varying conditions a reasonable basis is offered for the selection of the value of the coefficient of discharge.

EXPERIMENTS ON FLOW OF AIR

The results of the experiments made by Prof. R. J. Durley ("Transactions," American Society of Mechanical Engineers, Vol. 27, page 193) on the flow of air through orifices with pressure differences not exceeding 6 in. of water column (0.2 lb. per square inch approximately) are given in Table 2. The orifice in each case was formed by drilling a hole through a steel plate 0.0571 in. thick. The holes were cylindrical, with sharp edges on both sides of the plate. The channel of approach to the orifice was at least twenty times the area of the orifice, so that the velocity of approach was negligible.

TABLE 2—VALUES OF *c* FOR ORIFICES

Diameter of orifice, in.	Head in inches of water				
	1	2	3	4	5
5/16..........	0.603	0.606	0.610	0.613	0.616
1.............	0.601	0.603	0.605	0.606	0.607
2.............	0.600	0.600	0.600	0.600	0.600
3.............	0.599	0.598	0.597	0.596	0.596
4.5...........	0.598	0.596	0.594	0.593	0.592

It will be noted from this table that the coefficient of discharge decreases as the diameter of the orifice increases. It is probable that with small diameters the flow approximates that through a short tube, free contraction being more nearly obtained with the large diameters.

The results of the experiments conducted by S. A. Moss ("American Machinist," Aug. 10, 1905, Vol. 28, page 193) on the flow of air through a short tube 1 in. in diameter are given in Table 3. The average value of c determined from these experiments is 0.942. The length of the tube was one and one-half times its diameter, preceded by a rounded approach or mouthpiece having a radius equal to the diameter of the tube. Velocity of approach was negligible. The tube discharged the air into the atmosphere, the pressure varying from 4 to 8 lb. per square inch.

Table 4 is a brief summary prepared from extensive experiments on the flow of air through the valves of a gas engine as reported by Prof. C. E. Lucke ("Transactions," American Society of Mechanical Engineers, 1906, Vol. 27, page 232). Two types of poppet valve were used, one having a flat and the other a conical seat.

TABLE 3—VALUES OF *c* FOR SHORT TUBE

Pressure difference, lb. per sq. in.	Discharge, cubic feet per second		Coefficient of discharge
	Actual	Theoretical	
4	3.56	3.76	0.946
5	4.01	4.21	0.952
6	4.30	4.58	0.938
7	4.60	4.92	0.934
8	4.96	5.26	0.944

It will be noted that the coefficient of discharge in Table 4 is least for the largest valve opening, as noted in Table 2 for orifices. It should also be noted that the values of c correspond roughly to the values which might be expected for water flowing through the openings.

From a study of the effect of sharp edges, approach to openings, length of tube, circuitous passages, etc., it is thought that $c = 0.85$ should be the maximum assumed value for the coefficient of discharge of an air-inlet valve. It may be considerably less. It will probably lie between 0.60 and 0.95, as indicated by Tables 2 and 3. Unwin in his treatise on hydraulics, page 131, gives values of 0.64 for sharp-edged circular orifices, 0.81 to 0.83 for short cylindrical mouthpieces without rounding at inner edge, 0.97 for short conoidal mouthpieces and 0.86 for coned blast nozzles.

TABLE 4—VALUES OF *c* FOR POPPET VALVES

Pressure difference, inches of water	Diameter of valve opening, in.	Lift, in.	Area of valve opening, sq. ft.	Coefficient of discharge
Valves with Flat Seats				
1 to 30	1.58	0.05	0.001722	0.742 to 0.820
1 to 30	1.58	0.10	0.003444	0.673 to 0.690
1 to 15	1.58	0.15	0.005166	0.634 to 0.649
1 to 8	1.58	0.20	0.006888	0.600 to 0.612
Valves with Conical Seats				
1 to 30	1.5	0.05	0.001351	0.788 to 0.940
1 to 30	1.5	0.10	0.002391	0.927 to 1.027
1 to 15	1.5	0.15	0.00364	0.931 to 0.959
1 to 10	1.5	0.20	0.00494	0.855 to 0.872

The pressure-temperature curve for adiabatic expansion shown in Fig. 3 is obtained from the equation

$$T_1 = T_2\,(P_1/P_2)^{(k-1)/k} = T_2\,(P_1/P_2)^{0.286}$$

where T_1 = absolute temperature of the air when the pressure becomes P_1, and T_2, absolute temperature of the air at pressure P_2—taken as atmospheric pressure and 521 deg. Fahr.

The values obtained from equations (5), (8) and (10) are shown in Fig. 4 in graphic form, using a value of $c = 0.85$.

APPLICATIONS

The collapse of parts of the Bull Run pipe line of the Portland (Ore.) water supply is described in "Engineering News," Vol. 66, page 112. Twenty-two sections of lock-bar pipe having a diameter of 52 in. and thicknesses of 1/4 and 5/16 in. collapsed when a rupture occurred at a low level while the pipe was being tested. Another collapse of somewhat less extent occurred at another part of the pipe line for the same reason. A third collapse was caused while draining the pipe after a test.

In the first case mentioned there was a 4-in. air-inlet valve at the summit where the collapse occurred. The minimum size of air valve which would have prevented this collapse can be determined as follows: From Table 1 it is found that a vacuum of about 6 lb. per square inch is required to cause the collapse of a pipe 52 in. in diameter and 1/4 in. thick. The difference of elevation of the summit of the pipe and the point of rupture was approximately 225 ft. Since the maximum vacuum the pipe can withstand is about 6 lb. per square inch, we will assume that the air-inlet valve has been designed to maintain that pressure in the pipe. The net head causing the water to flow from the ruptured section of the pipe was therefore about 211 ft. The length of the pipe between the summit and the rupture was about 2400 ft. Assuming that the rupture was large enough to cause no obstruction to the flow, the maximum velocity in the pipe is found from equation (4) to be $u = 33$ ft. per second. With this value of u and with 6 lb. per square inch vacuum, the required area of air-inlet valve is found from Fig. 4 to be 0.042 times the area of the cross-section of the pipe. That is, a single valve having an effective area equal to the area of a 10.7-in. circle will be required when the coefficient of discharge is 0.85. At least eight 4-in. or four 6-in. air-inlet valves would have been required to prevent the collapse of this pipe.

By the same method a minimum of five 4-in. air-inlet valves would have been required to prevent the collapse of the other part of the line when the rupture occurred there.

It was noticed in connection with the collapses mentioned above that those parts of the pipe line which had been covered with earth did not collapse, even though they were above the parts which failed. The earthfilling can resist for short times the forces which tend to collapse the pipe.

The Engineering Record of July 22, 1912, page 92, has a description of a 15-ft. riveted steel pipe line at Grand Falls, Newfoundland. A 36-in. riser pipe was placed at the summit to allow air to enter and prevent collapse. The thickness of the pipe was not given, but assuming it to be 3/8 in. it will be seen from Table 1 that a vacuum of less than 1/2 lb. per square inch will cause the collapse of this pipe. The area of the riser pipe is 0.04 of the area of the cross-section of the penstock. From Fig. 4 it will be seen that a velocity away from the summit of about 10 ft. per second would cause the collapse of this pipe, on the assumption that the coefficient of discharge of the riser pipe is 0.85.

Investigation of Electrolytic Iron

TRYGVE D. JENSEN of the electrical engineering department of the University of Illinois has for the past two years been making a special study of electrolytic irons with reference to their magnetic qualities. The aim has been to secure an iron of high permeability and small hysteresis loss and an iron four times more efficient than any now in use has been secured. A vacuum furnace was used for melting the iron, the pressure in the furnace being reduced to about one-half centimeter of mercury. Pure iron, when melted in the open air, will absorb a quantity of carbon and oxygen and so be of inferior quality, while iron melted in a vacuum does not absorb any of these elements but remains pure. Various methods were used in cooling, tests being made to determine the effect of cooling slowly, plunging in brine, in liquid air, etc., each iron being tested to determine the magnetic qualities, metallurgical composition, chemical composition, strength, and critical temperature. Much valuable information has been gained by these experiments, the results of which are soon to be published in the form of a bulletin by the engineering experiment station of the university.

THE TAPPING of a 60-in. steel main in Pittsburgh was described in the Engineering Record of April 18, page 453. The article was in error in giving the size of the taps as 36 in. All three of them were 24 in. in diameter.

Improved Asphalt Penetrometer

Electrically Controlled and Timed Device in Use in the New York Testing Laboratory

ADDED accuracy and reduction in the time of operation are advantages claimed for an electrically controlled and timed asphalt penetrometer which has been in use at the New York Testing Laboratory for over a year. The apparatus, which has been developed from the Dow penetrometer by Hermann W. Mahr, engineering chemist at the laboratory, was described by him in the February issue of the "Journal of Industrial and Engineering Chemistry," from which this abstract has been taken.

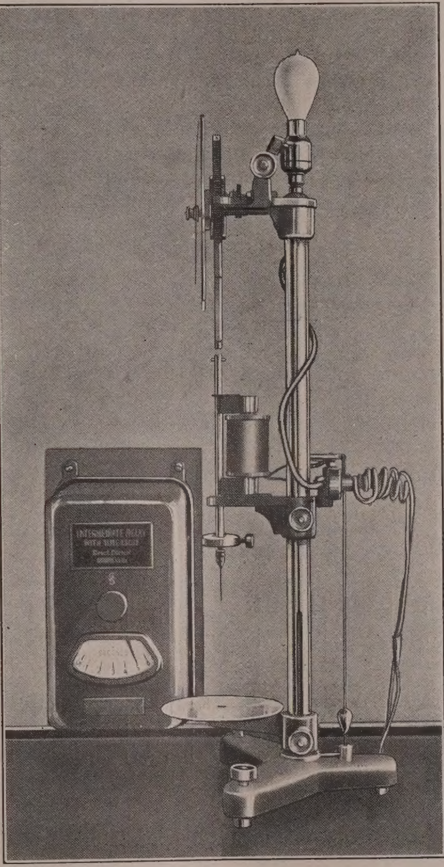
The original innovations were made in an effort to correct certain characteristics of the Dow machine which necessitated a long series of observations to afford accurate results. As some specifications allow a range of only 10 deg. in penetration, the field for the improved device is apparent. The Dow type consists essentially of a needle of specified size fixed in a rod, the rod and needle being loaded to a definite weight. The rod is held with the needle by a clamp, which allows the needle to penetrate with a minimum friction. A device for measuring the amount the needle has penetrated after it has been released for a specified time and again grasped by the clutch is also necessary.

Features of New Instrument

In the Dow instrument the shelf, on which rests the water bath containing the sample, is fixed in position, limiting the height of the vessel which contains the sample and causing loss of speed and accuracy in adjusting. The shelf on the improved instrument is moved on a screw, which allows the sample to be brought up to the needle very slowly. Another change has been made in placing a weight on the lower part of the rod just above the needle, eliminating a weighted rectangular aluminum frame which was an inconvenience of the older instrument due to its bulk. The rod which holds the needle has been made to slide in a collar of considerable height and is thus maintained in a vertical position, which is impossible in the Dow instrument because of the weight of the large aluminum frame and the fact that when the clamp holding the needle is released the space within is quite large. The improved clamp was fixed in the long collar, and, instead of by means of a counterweight, the rack was kept in position by a spring pressing against it.

Recognition of the fact that the greatest personal errors in making determinations with the machine lay in setting the rack on the rod before and after penetrating and in timing the penetration period, however, led to the introduction of the electrical control. Formerly timing was done with either a stop watch or a metronome, and the instrument was subject to shocks due to the manual control. In 1894 Dow and Griffith patented an electrical limiting-time-interval clamp, which, however, was later discarded because of its cumbersomeness and complication. The advent of the simple, compact Sieman intermediate relay time limit has made possible the direct control of the penetrating device by the electrically timed magnet, instead of through a clamp.

The electromagnetic clutch now used in the New York Testing Laboratory for holding the rod with the needle is on a bracket sliding on the upright standard of the instrument, to which it may be secured by means of a setscrew. The rod which holds the needle is of steel, brass-clad, which slides through openings in thin German silver plates placed outside of the concave poles of the electromagnet. These poles partly enclose and guide the rod. The plates are so set that the rod does not come into contact with the magnet's poles, and are rounded where they touch the rod, thus eliminating friction.



Improved Penetrometer

The penetration is measured, as is usual, by means of a rack and pinion, the latter being fastened to an adjustable hand on a dial. The counterweight and spring used in previous types of instruments for holding the rack in place have been dispensed with. The pinion shaft has an additional gear wheel. This gear is in mesh with a worm on a shaft at right angles to the first, the worm shaft being driven by means of a milled head at one side of the dial. By turning the milled head the rack may be raised or lowered and accurately set on top of the rod. In setting the needle on the surface of the sample, the latter is first raised to within less than a millimeter of the former. The rod is then forced down by the rack until the needle touches the surface. During the setting the rod is held by the magnet, the electromagnetic force being overcome by that exerted by the rack. This device and method of procedure enable very accurate settings of the needle and rack. A plumb bob and adjusting screws are attached to the penetrometer, to aid in setting the machine in the vertical position. A small electric lamp at the top of the standard, in series with the magnet, indicates when the current is on. The instrument is wound to be connected with a 110-volt direct-current circuit, which may be broken by pressing a button. The penetrometer is placed in the circuit with one side of the Sieman relay, and the lighting circuit is passed through a switch and the relay electromagnet. On breaking the current through the latter, the penetrometer circuit is broken and automatically made at the end of five seconds.

Accuracy Tests

Tests of the penetrometer's accuracy were made by determining the penetration of three asphaltic cements, using the same needle. Three determinations were made for each sample, under the usual conditions at 77 deg. Fahr., with the needle loaded to 100 grams, penetrating for five seconds. After each observation the needle was cleansed with chloroform, dried and brought again to 77 deg. Fahr. Two samples of oil asphalt cement were tested, each set of three giving uniform penetrations—55 and 53 respectively. The results for a Trinidad sample were 85, 84 and 84. The close agreement of all these determinations is the best indication of the worth of the improvements which have been introduced.

First Link of Projected India-Ceylon Bridge

A VIEW is shown on the front cover of the 289-ft. Scherzer rolling-lift span forming part of the bridge from the main land of India to the island of Rameshwaram. This bridge, as stated in the issue of May 2, page 506, is the beginning of what may ultimately connect India with Ceylon, along the rocky reef known as Adam's Bridge. Such a connection would be 22 miles long.

The bridge already built is made up of 143 40-ft. and two 44-ft. plate-girder spans, in addition to the rolling-lift span. The piers are of granite masonry with concrete footings in steel caissons. Some of them were sunk by the pneumatic process through an artificial island 60 ft. square, made with a timber caisson filled with coral boulders and sacks of concrete covered with sand and clay.

The bridge was erected by natives from the adjacent mainland, who became so expert that the riveting squads drove 250 rivets each per day with pneumatic hammers. The steel was hoisted by hand windlasses and the two leaves of the bascule span were erected in inclined positions with the trusses counterweighted for stability. The segmental girders were handled by gantries and the bascule trusses were erected as cantilevers, without falsework, by stiffleg derricks advancing upward on the top chords.

Concrete for the counterweights was mixed in a ½-yd. machine mounted on a pontoon. The machine dumped on the deck of the pontoon, whence the concrete was shoveled into galvanized iron dishes about 12 in. in diameter and 5 in. deep. These were carried to the forms by young coolie boys and coolie women of all ages, who deposited it at the rate of 2 cu. yd. per hour.

THE HOOPED COLUMN formula on page 509 of Ernest McCullough's article, "Girderless Reinforced Concrete Slabs," in the Engineering Record of May 2 was printed incorrectly. It should have read

$$P = f_c\,(A_c + nf_s + 2.4\,nf_h)$$

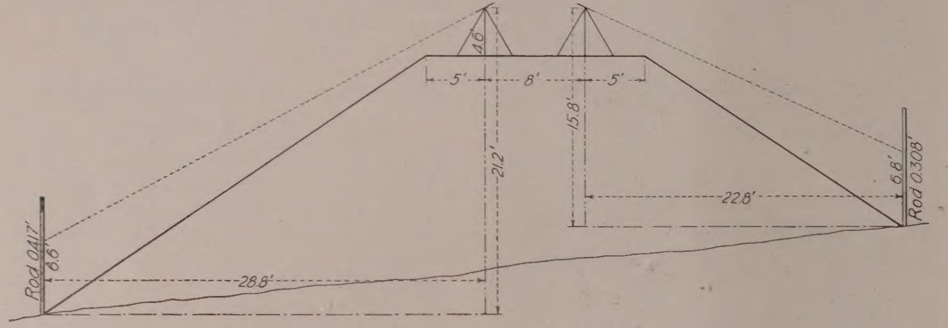
In the second column on page 509, under the side heading "Column Capital," the angle for the slope should be 45 deg. instead of 55 deg.

New Instrument for Measurement of Earthwork

Application of Special Stadia-Arc Graduations to Facilitate Cross-Sectioning Railroad Embankments for Valuation Purposes

MEASUREMENT of earthwork in railroad fills and cuts constitutes a large part of the field work in the Federal evaluation of railroads. The work involves innumerable cross-sections, and the importance of any special means of expediting the work or minimizing the computations is apparent. An instrument has been devised which entirely eliminates the need for trigonometric tables in the stadia work and reduces the calculations to simple multiplications and additions. The instrument is stripped of all attachments not needed for this work alone and is designed to permit the most rapid use.

A view of the instrument is shown herewith. The telescope is that of the transit, with stadia wires. The plate on which the telescope is mounted is fitted to a Johnson head, which is leveled by sliding on a concave surface and clamped by the upper winged nut underneath. The whole moves in azimuth and is clamped by the lower nut. There are one circular level, one tubular level parallel to the plane of sight, and a single leveling screw in the same plane. The only scales are on the vertical quadrant, which is an adaptation of the Beaman stadia arc, devised several years ago for application to the vertical circle of the ordinary transit. The graduations are placed on the broadened circumference of the arc, however, to allow the instrumentman to read them without change of position.



Station	Direction	V.Arc	Intercept	Rod	Dist apart of Settings	H I	H.Arc	Width of Roadbed	Vertical	Horizontal	Volume	Remarks
246	Left	4	0.417	6.6	8	4.6	31%	18	21.2	28.8		
	Right	6	0.308	6.8		4.6	26%		15.8	22.8		

Method of Taking Cross-Section and Sample of Field Notes

GRADUATIONS

These graduations are figured to give simple multiples of the rod interval, thus determining rapidly and without intricate calculations the precise difference in elevation and corrected horizontal distance. One row, marked V, has its zero figured 50 and the algebraic sum of the number indicated and —50 shows by its sign whether the telescope is elevated or depressed. This sum is the multiple for the rod readings that determine the difference of level between the instrument and point on the rod cut by the middle wire. The other arc is so spaced as to read the percentage to be deducted from the observed distance to obtain the true horizontal distance.

In using the instrument the tripod is set up in the middle of the roadbed, if this position will command a view of the rod at the four governing points on the two slopes. For multiple tracks on high fills a separate setup may be needed near each shoulder, in which case the distance between settings must be noted. In setting up, the plate is leveled by the circular level and clamped. Orienting on a rod held plumb in a line at right angles to axis of the prism to be measured, the instrument is clamped and leveled more carefully by the tubular level, using the single leveling screw. This establishes the zero which governs all readings.

With the index of the arc coinciding with any division whatever, provided the stadia wires are on the rod, readings are taken of the arcs V and H, the stadia intercept and the middle-wire reading. Similar readings must be made for the other slope, the width of roadbed must be noted, and if two setups were required the distance between them must be recorded. This constitutes all the field work for one station.

The accompanying sketch and sample of field notes indicate the facility with which the work can be done. The illustration applies to a fill, but the method is equally applicable to a cut. In the office the V-arc reading —50 is multiplied by the stadia intercept, and the difference between rod reading and height of instrument is added to obtain the height of cut or fill. For the left-hand slope illustrated

$$(4 - 50)\ 0.417 + 6.6 - 4.6 = -21.2$$

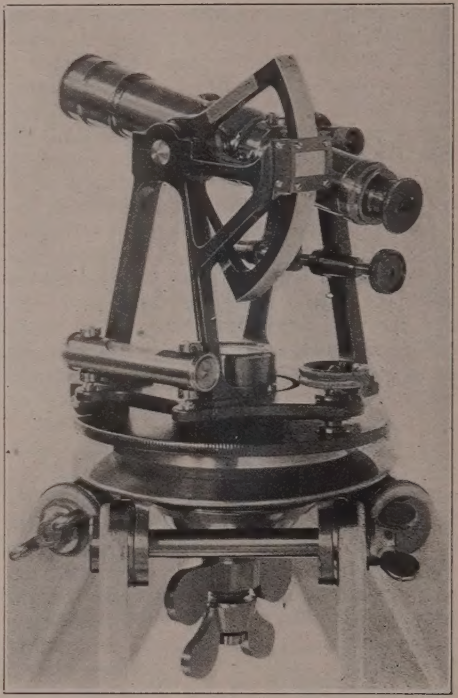
To get the correct horizontal distance, the observed distance is multiplied by the percentage indicated by the H arc, and this product is subtracted from the observed distance. For the left-hand slope illustrated

$$41.7 - (41.7 \times 0.31) = 28.8.$$

With the right-hand figures computed in a similar manner, either the area of the section or the volume of the prism may be readily determined.

The instrument is made by W. & L. E. Gurley, of Troy, N. Y., who are the sole manufacturers of the Beaman stadia arc. It was specially designed for use on the Federal valuation work, and a number of the instruments have been furnished to the engineering board of the Interstate Commerce Commission and are being used on the work.

Since the above description was prepared information has been received concerning a stadia circle recently perfected and put on the market by the Keuffel & Esser Company, of Hoboken, N. J. Like the original Beaman circle, the K & E stadia circle is an addition to the vertical circle of the ordinary transit. It also serves the same purpose as the Beaman circle, the graduations being similar; the notations, however, are different. On the K & E attachment what corresponds to the V scale reads both ways from zero horizontal and the H scale reads down from 100 instead of up from zero. These notations give direct multipliers and still further simplify the calculations. In the example given above the vertical scale would read 46 and the difference of elevation sought would be $-46 \times 0.417 + 6.6 - 4.6 = -21.2$, while the horizontal scale would read 69 and the corrected horizontal distance would be $41.7 \times 0.69 = 28.8$.



New Cross-Sectioning Instrument

Structural Shapes Produced in 1913

FIGURES issued by the Bureau of Statistics of the American Iron and Steel Institute show that 3,004,889 gross tons of iron and steel structural shapes were produced in the United States in 1913, an increase of 158,402 tons, or 5.5 per cent more than in 1912. Of this total, 2,553,806 tons —an increase of 3.3 per cent—were heavy structural shapes and 451,083—an increase of nearly 20 per cent—were light structural shapes. The figures exclude plates, merchant bars, reinforcing rods and sheet piling. Pennsylvania made 71 per cent of the total production, as compared with 72 per cent in 1912. The next largest producers in both years in the order of their prominence were Illinois, Indiana, New York, Wisconsin, Ohio, Alabama, California, New Jersey and Colorado. The total production of iron and steel wire rods was 2,464,807 gross tons, a decrease of 7.1 per cent. Nails and spikes cut from plates decreased 13.9 per cent.

Letters to the Editor

Bonding Asphalt to Concrete

SIR: In building the concrete reservoirs which have been constructed recently in San Francisco it was desired to apply tar or asphalt coatings in several instances, and the problem of bonding asphalt to concrete had to be solved. After making unsuccessful attempts to apply the asphalt direct to the concrete in various ways, the expedient of first painting the surface with coal tar was tried, and it was found that when applied in this way under right conditions the asphalt would adhere so firmly that after it had cooled it could not be broken away without bringing pieces of concrete with it.

To secure this result the concrete surface was first scrubbed clean with a bristle brush, and over this a thin coating of hot coal tar was "painted." The coal tar should be heated in small quantities, brought just to the boiling point and then applied immediately. Heating the coal tar in large quantities, which necessitated some delay before it could all be used, did not give such good results, and the gangs were therefore not allowed to heat more at a time than they could apply quickly as soon as it began to boil. The asphalt was spread over the tarred surface in the usual manner.

San Francisco. B. N. ABBOTT.

Marketing of Engineering Materials

SIR: Trade conditions involving materials used in engineering work, while seemingly becoming better each year, are still far from what they should be. This view is supported by the recent discussions in the Engineering Record regarding timber, but this class of building material is not the only one which suffers from the effect of improper marketing methods. In fact, practically all materials are affected more or less and the intention of this letter is to show, if possible, where some of the weak spots lie.

While we may say in a general way that the methods of marketing most materials are far from perfect, for the purpose of illustration let us consider these methods as applied to the field of reinforced concrete. Here especially are conditions to be deplored. Manufacturers are continually placing on the market various kinds of reinforcing material and other devices, some of which possess much merit but many of which, according to the manufacturer, possess marvelous powers when embedded in concrete. As a matter of fact if these materials upheld the claims made for them they would defy all the laws relating to the strength of materials. The use of such material has been the cause of many failures, and every failure in any particular type of construction reflects on that class as a whole, the good materials suffering with the poor. This obviously shows the necessity of bettering trade conditions, and by doing so the unscrupulous manufacturer is going to be eliminated, thus decreasing failures and increasing confidence in this field of engineering.

When an advancement is made in any line of engineering construction, unscrupulous manufacturers are quick to take advantage of it by producing a product which, if not investigated thoroughly, has all the ear-marks of being equal to the materials used by the originator of the improved construction. The main thought in a case of this kind is the perfection of the sales department at the expense of the engineering department. A shrewd salesman is able to unload much of this product on the confiding public, which, in a great many instances, does not think it necessary to employ the services of an engineer who is capable of analyzing the results. As I have already stated, some of the reinforcing materials possess much merit and deserve recognition, but, in my opinion, a great deal of it is placed on the market with claims for strength and efficiency which are totally unwarranted. Many of them die a natural death, but often not until the defects have shown up in the form of failure of the structure, causing financial loss and in many cases loss of life.

I am inclined to believe that much better trade conditions would result if the general public understood better the real work of the engineer or architect and could see the advantage to be gained by employing men who keep in touch with the progress in engineering and who, through practical experience and study, are able to determine the proper materials to use in engineering work. The engineering societies can do much to educate the public along this line, and it is to be hoped that more effective steps will be taken in the near future than have been in the past to bring this about.

In order to show some of the methods still employed by salesmen, I will state the case of an owner who contemplated the erection of a reinforced-concrete building and who considered the services of an architect or engineer wholly unnecessary. The salesman, who was connected with a firm which sold patented deformed reinforcing rods, got in touch with this owner and persuaded him to go to New York City for a week's pleasure trip. The steel was sold to him for his proposed building even before plans had been made, and the owner upon his return home told a contractor what a great time he had had and the best part of it was that the trip had not cost him one cent. The contractor, after finding out the price that had been paid for the steel, was able to show him how he could have saved money by taking that trip alone and buying the steel at the market price. Although this is rather an unusual case, it serves to illustrate what measures are sometimes taken in order to effect a sale.

The proper solution of the present problem involving trade conditions cannot be reached without much study. As food for thought, however, why is it not feasible to produce an organization, similar to the Board of Fire Underwriters, which could provide a testing laboratory where the various materials on the market worthy of consideration could be tested? Why would it not be advisable for engineers to refuse to consider the use of materials which had not been thoroughly tested by uninterested parties and from whom he could obtain the results of their tests? Thousands of dollars are spent needlessly each year in conducting tests that duplicate work done previously. This money would establish and support a laboratory of considerable magnitude. If such a scheme as this could be carried out manufacturers would be obliged to submit their materials for tests in order to have consideration from purchasers, and if given a favorable report no better recommendation could be asked for.

Some may argue that an arrangement of this kind would give an undue advantage to some producers, but it should be possible in every case to purchase engineering materials strictly according to merit and not according to the merits of the sales department.

All manufacturers have their own handbooks giving tables of strength and other qualities. Tests from which these tables are derived are usually made by manufacturers, some of whom are inclined to stretch things a trifle, and the tests are not always representative of average conditions. If they published results of tests made by uninterested parties who would consider the material from all standpoints, there would immediately be shown an increased confidence in materials which are really deserving of credit, while the poor materials would be eliminated.

The engineer's position at the present time is such that he insists on using only such materials as he is thoroughly familiar with, rather than accept any statement of the manufacturer of new materials regarding his products, the use of which might effect a considerable saving and perhaps provide a structure more suitable to the requirements.

May we hope to have the views of other engineers regarding the trade conditions involving engineering materials, trusting that any discussions produced may result in a material advancement in the methods of marketing such materials.

J. J. MORGAN,
Columbus, Ohio. Consulting Engineer.

How the Autoclave Cement Patent Was Granted

SIR: I have noted very carefully the articles contained in your issue of May 16 under headings, "Patent Office Procedure" and "How the Autoclave Cement Patent Was Granted," and beg to be permitted to come to the defense of the Patent Office.

While returning from the cement show at Chicago in February, I became especially interested in the references to the issue of the autoclave patent then contained in the current issues of Engineering Record. I felt some surprise also at the grant of the Force patent and some days after my return to this city I took the trouble to make a review of the file record of the case, thinking that I might be able to throw some light upon the situation involving the grant of the patent. Owing to the fact that a work relied on by the patent examiner as a basis for rejecting the Force application did not happen to be accessible at the time of my examination of said record, I was delayed until recently in completing my examination. Meanwhile your representative appears to have preceded me.

It is hardly necessary to refer additionally to the course of prosecution of the Force application in the Patent Office. The disclosures of your correspondent in your issue of May 16 suffice to indicate that the examination of the application was anything but exhaustive, and as a patent expert, I believe that there is quite a fair presumption that the Force patent, if ever relied on as a basis for an action of infringement, will be declared invalid, and if not invalid so restricted in its scope as to be practically valueless. I fail to see wherein the limitation of the claims of this patent to certain definite pressure conditions can be reasonably maintained to establish patentable novelty, having in view the state of the prior art comprising references to certain publications, an old German patent, and certain commonly practised expedients in cement testing. The patentability of the Force invention, I may properly state, will depend in the final analysis upon whether the patentee may be able to maintain that he secures a new result, not merely greater efficiency owing to the fact that he subjects the cement in the treatment of the particular pressures, definitely prescribed by his claims. Were this a possibility, something that appears offhand to be unlikely, the scope of the patentee's claims must indeed be greatly restrictive. Those versed in the different methods of treating cement for testing purposes will be enabled to formulate a very excellent idea of the amount of invention exercised by this patentee in evolving the process under consideration, so I need not say anything further in this matter.

Passing now to the procedure in the Patent Office, I am constrained not to attempt to excuse with too much force the apparently inadequate examination of the Force application. It is somewhat remarkable to me that any actions should have been taken by the Patent Office based upon arguments or explanations orally submitted by the applicant and which were not made of record in the application. It is almost the universal rule of the examiners of the Patent Office to require all arguments at oral interviews submitted by an inventor or his counsel to be spread fully upon the record so that the public may be advised as to the position of the Patent Office in the grant of patents. This rule certainly is very rarely excepted to. I feel, however, that there are two main things which may be ascribed as the causes for the occasional grant of a patent such as that for the autoclave test. Certainly it is remarkable in my judgment that the Patent Office is so highly efficient that so few invalid patents, or patents of doubtful validity, are granted at the present time.

The first condition adverse to the highest efficiency in the Patent Office which I have in mind is the fact that the examining force of the Patent Office is inadequate to handle the work. Editorially Engineering Record properly lays the blame for this condition at the doors of Congress. Commissioner after commissioner entrusted with the administration of the patent laws has urged the necessity for increases in the corps of examiners. I believe that it is recognized that so far as individual efficiency and capability are concerned, the men who constitute the examining corps of the Patent Office are equal, if not superior, to any engaged in this class of work in the Patent Offices of the world. Certainly it is true that no other Patent Office examining corps is required to work under the pressure and under the same conditions as our patent examiners, and I refer particularly to the lack of room in the Patent Office, necessitating the crowding of these men engaged in work requiring the highest degree of concentrated effort in small spaces or in rooms poorly lighted and often unsanitary. It is a marvel to me that the patent attorneys of this country do not make a more concerted effort in co-operation with such highly useful and influential mediums as Engineering Record to convince our Congress that the time has come for the entire reorganization of the Patent Office on a footing of up-to-date efficiency.

The second condition detracting greatly from the operation of the Patent Office to best advantage is the incomplete classification of patents, of which there are nearly eleven hundred thousand issued in the United States alone, and more than twice as many in all foreign countries together, not to mention at all the tremendous number of publications not including patents, but which are just as effective references in the anticipation of invention. With less than 50 per cent of the patents granted in this country, and this work suspended at intervals owing to inadequate appropriation, certainly our Congress should act upon the urgent recommendation of the present commissioner, Hon. Thomas Ewing, for the increase of the classifying force. In this connection the following quotations from the last annual report of Commissioner Ewing are most appropriate and bear directly upon the situation involving the issue of the Force autoclave test patent.

"The primary function of the Patent Office is to ascertain and apply properly to any application for a patent all existing patents and literature which have a legitimate bearing adverse to the claims presented. To do this with any degree of success, it is essential that the American and foreign patents and the literature of the various arts collected in the scientific library should be classified and indexed so as to enable the examiners rapidly to find all existing patents and other publications bearing upon the application being searched. The system of classification which grew up in the office prior to 1898 has been found to be no longer adequate, and since that date a force of classifiers has been engaged in an attempt to work out a new and improved system of classification. Out of 1,085,000 United States patents, 517,000 have been assigned to their proper places under the new classification. To complete the classification within three years would require an addition to the examining force of seventy-five men. Meanwhile the delay in the completion of the classification work impairs the work of the Office to a serious extent."

"Our valuable library is in present need of reorganization and enlargement. The collection of books, periodicals and foreign patents is of the utmost value to the examining corps, inventors, manufacturers, and attorneys interested in investigating the novelty of inventions, but the material is not and never has been in condition for ready reference. Much of it is practically unavailable. I have started the work of reclassifying and indexing, with a view to meeting more nearly the peculiar needs of those who use the library. This library of about seventy-five thousand volumes of scientific works, about two million foreign patents, used, as it is, by people who are doing work which is almost identical with that of the examining corps, requires a librarian of special training and ability."

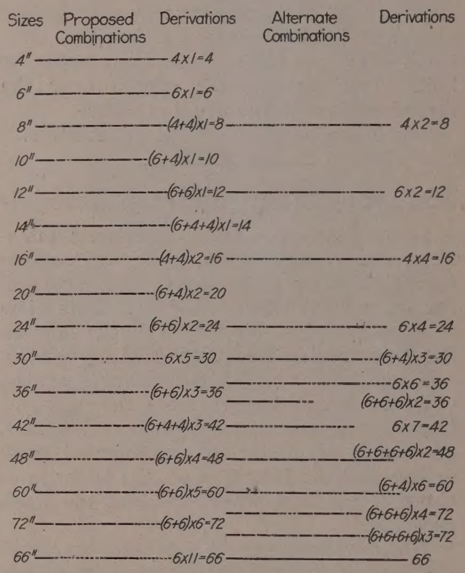
In conclusion I might say that it is a startling fact that in my own practice I have noted that it is not infrequent that the German Patent Office has cited prior patents out of the United States patent art which more closely affect the novelty of inventions than do the patents cited from this art in our United States Patent Office. Is this not a deplorable condition indeed and one which should be immediately remedied? A remedy is the building of a new Patent Office of a size and equipment suitable to handle the protecting of inventions of the greatest inventive people in the world, and the operation of said new Patent Office on a thoroughly efficient basis, all of which may be effectuated by merely using the nearly seven and a half millions of dollars actually earned during the operation of our Patent Office over and above the expenses of its maintenance.

Washington, D. C. J. F. ROBB.

Simple Method of Showing Sizes of Pipe Lines

SIR: The accompanying sketch indicates a simple method of so indicating pipe lines or other objects that are symmetrical but of different sizes that the size will be seen at a glance. This is not easy to do when there are many different sizes and each size is represented by a symbol taken at random, with no other thought than to provide a character different from those used for other sizes.

By the method here suggested the only symbols necessary to keep in mind are the long dash, representing 6 in. of diameter, the short dash, representing 4 in. of diameter, and the dot. The sketch is self-explanatory. The number of dots is the multiplier of the sum of the lengths represented by the dashes in one group. Thus all sizes can be indicated by various combinations of these three symbols.



System for Showing Pipe Lines

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OCCUPATIONS OF ENGINEERING graduates are analyzed in the forthcoming alumni register of the College of Engineering at the University of Illinois. The figures show that 89 per cent of the 2165 graduates are engaged in some kind of engineering work, and that only 173, or about 8 per cent, have gone into other fields. Architects, executive officers of engineering companies, and teachers are classified with consulting engineers and employees of engineering companies, making a total of 1933. The other occupations classified include 52 farmers and ranchmen, 31 merchants, 19 executive officers of mercantile companies, 19 editors, and 8 army officers.